VOL. XXXV. No. 10. OCTOBER, 1957

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THE STRUCTURAL ENGINEER

THE JOURNAL OF THE
INSTITUTION OF STRUCTURAL ENGINEERS

The President, 1957-58

The Design and Construction of Certain Temporary Works, The Upper Silent Valley Reservoir, Co. Down
by W. J. Shirley (Member)

The Design and Construction of the Prestressed Concrete Hangars at Helsinki Airport, Finland
Discussion on the Paper by Dr. F. W. Gifford (Associate-Member) and Mr. Matti Janhunen

Structural Problems Connected with Palletisation
Discussion on the Paper by Mr. Brian Scruby (Member)

Post-War Developments in German Steel Bridges and Structures
Discussion on the Paper by Mr. G. B. Godfrey (Associate-Member)

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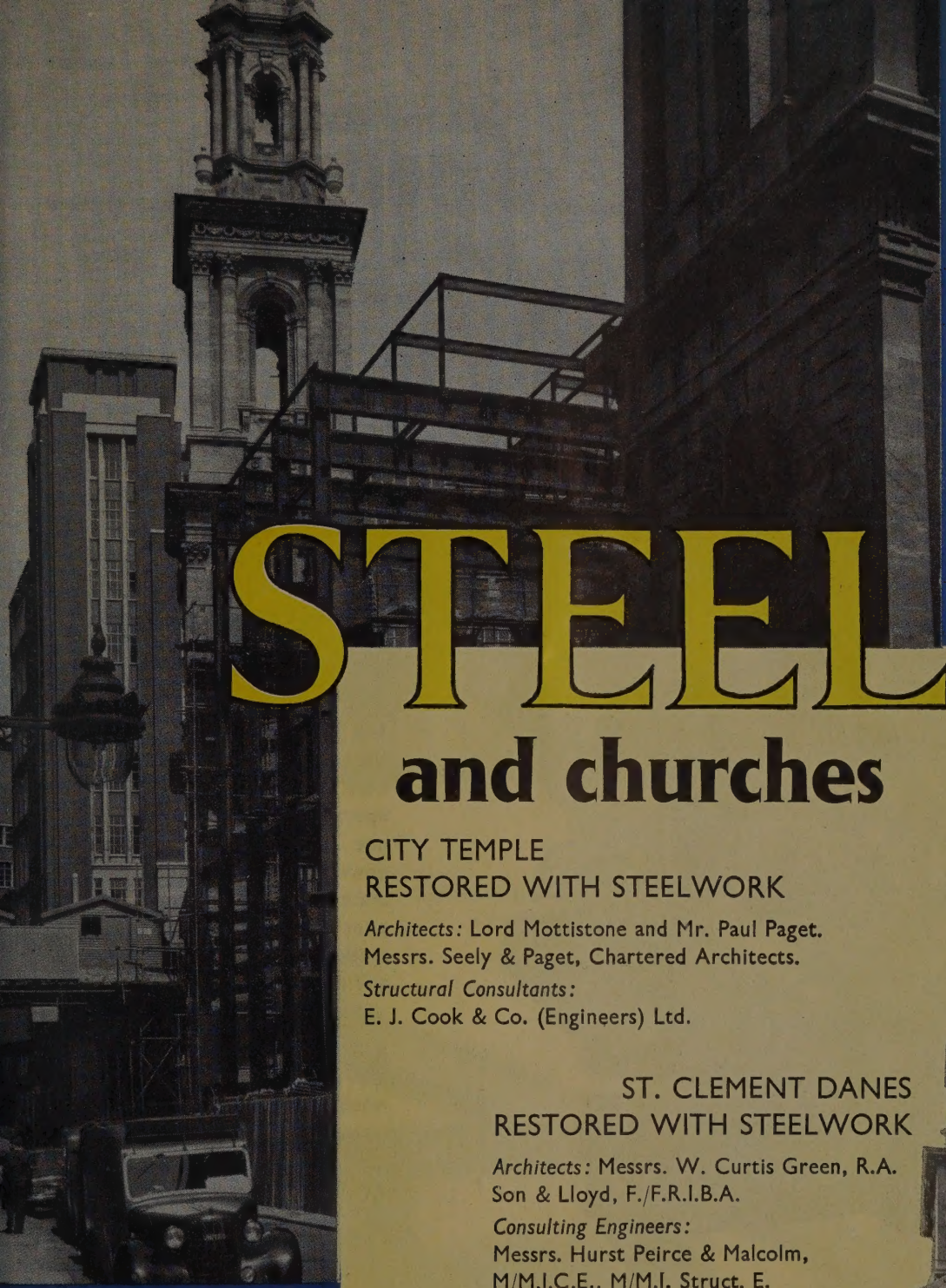

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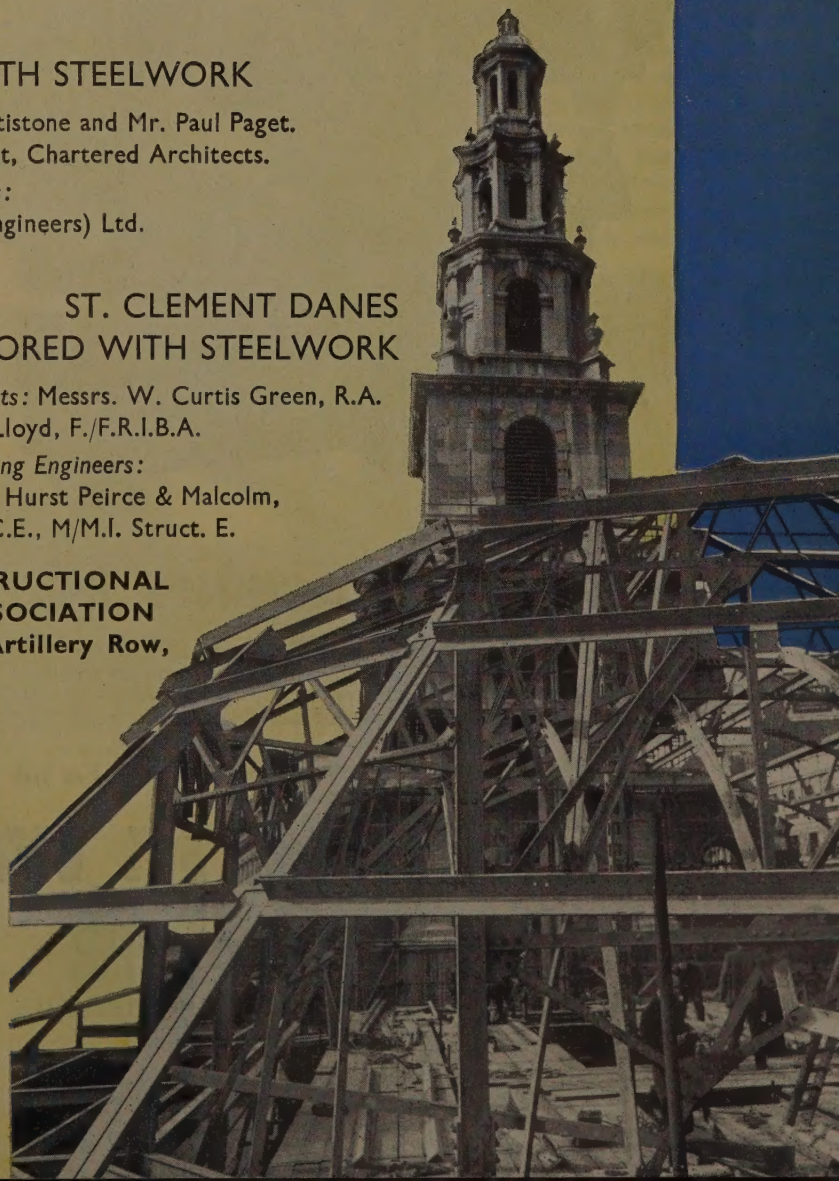

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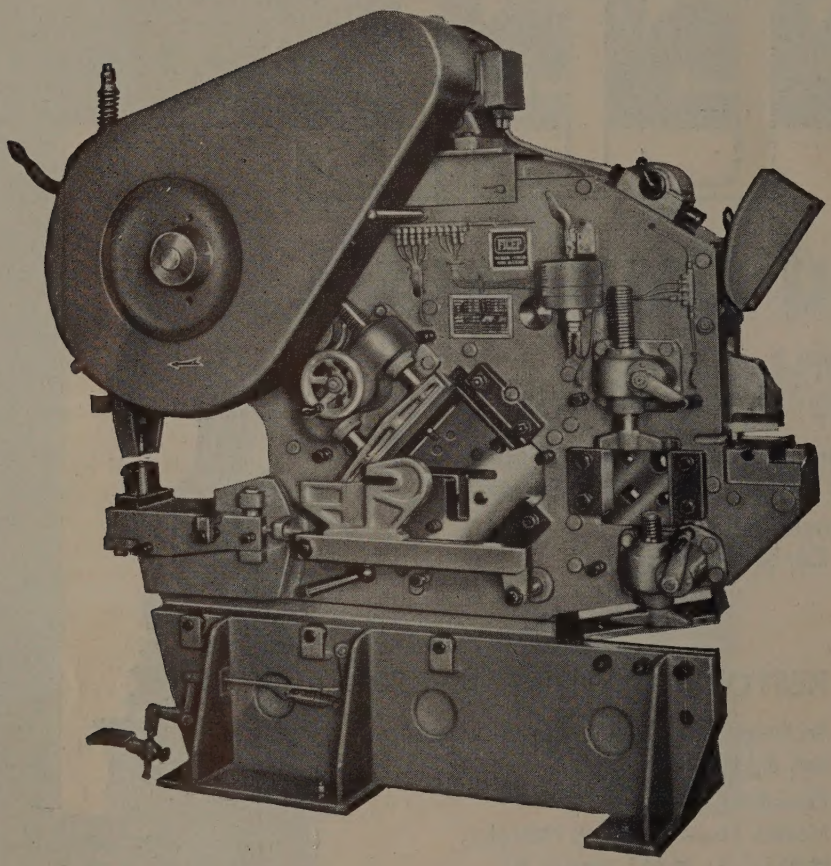

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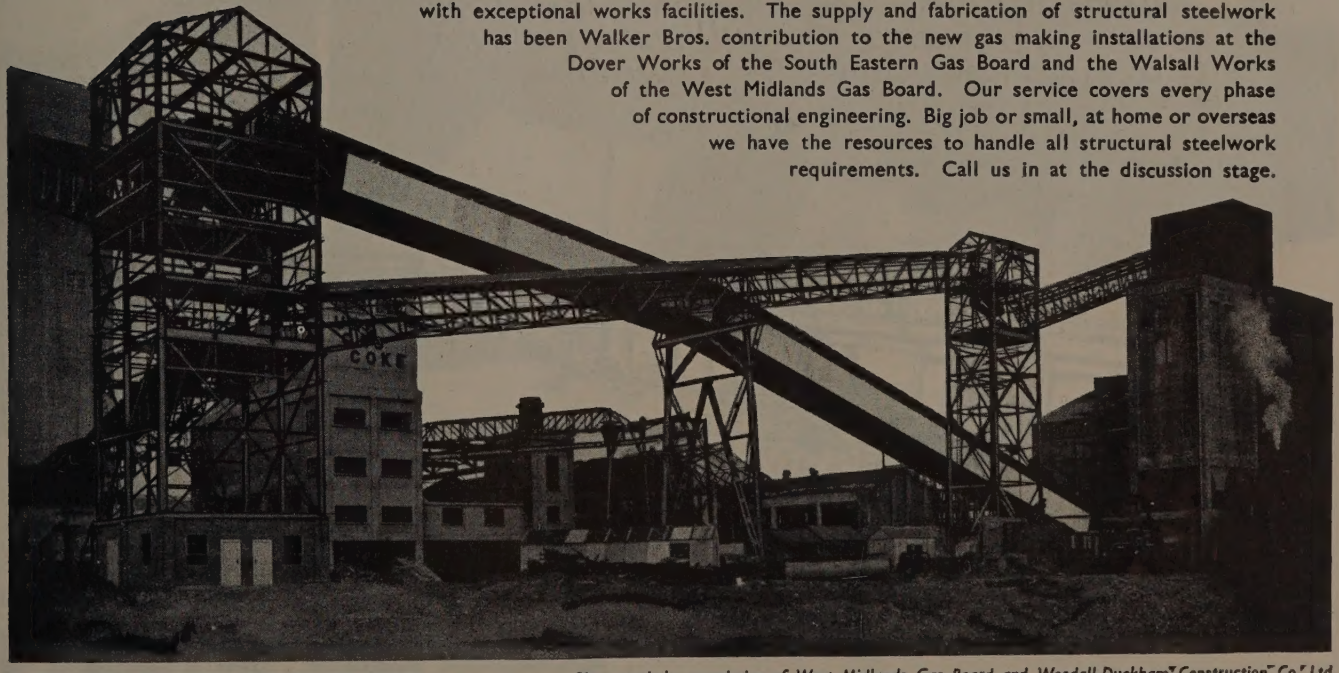

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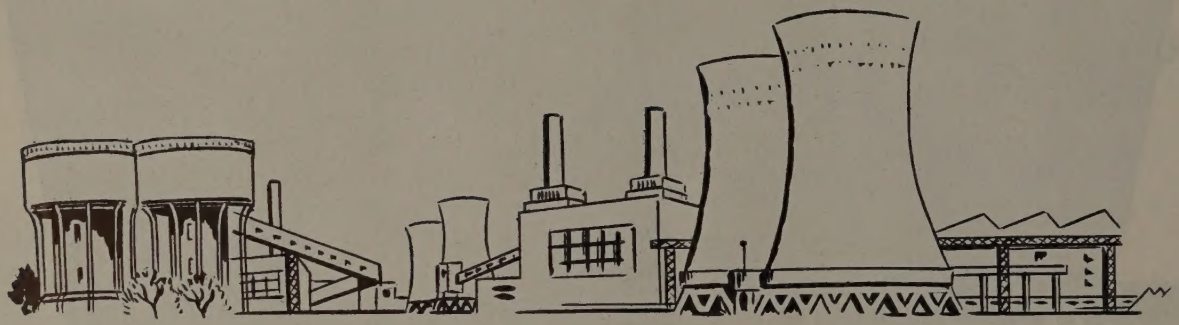

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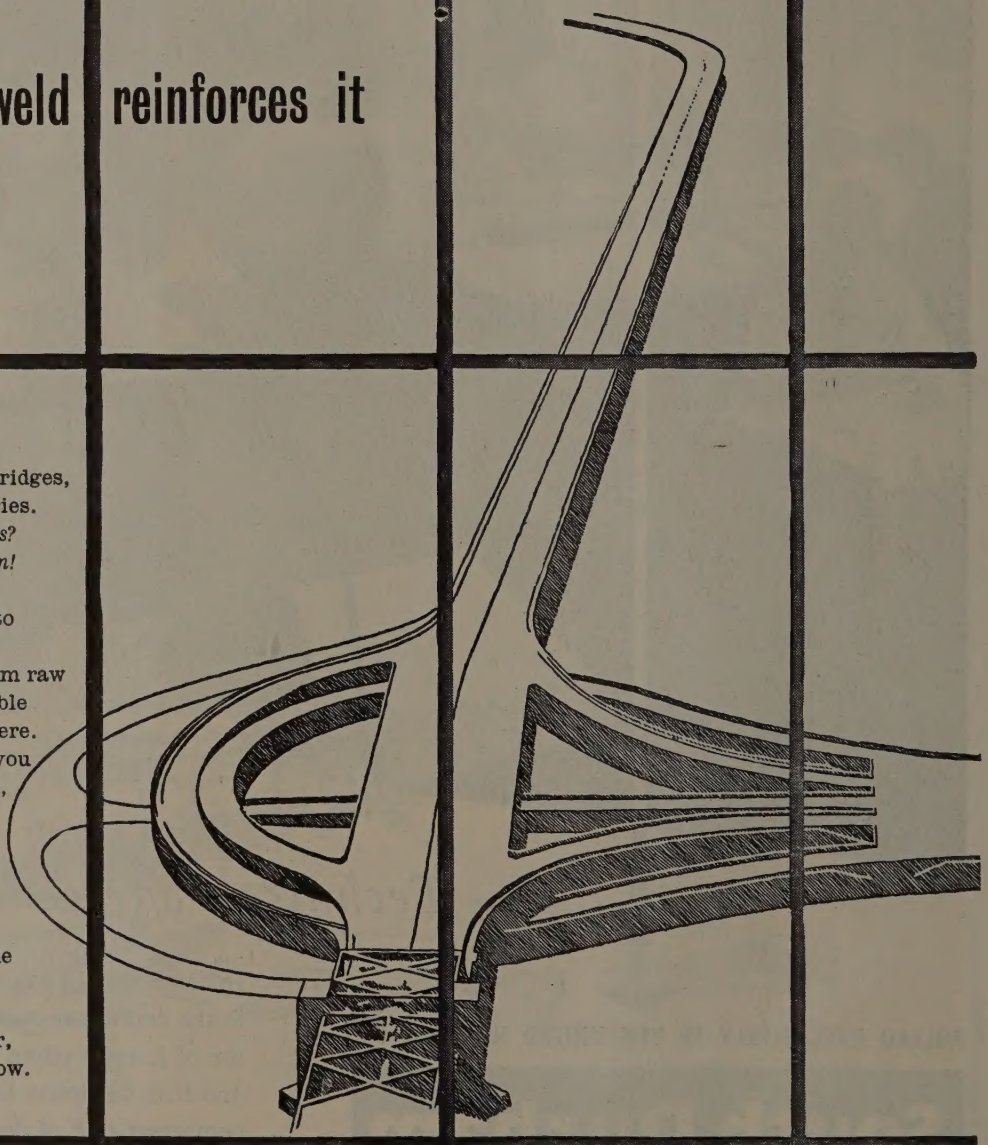

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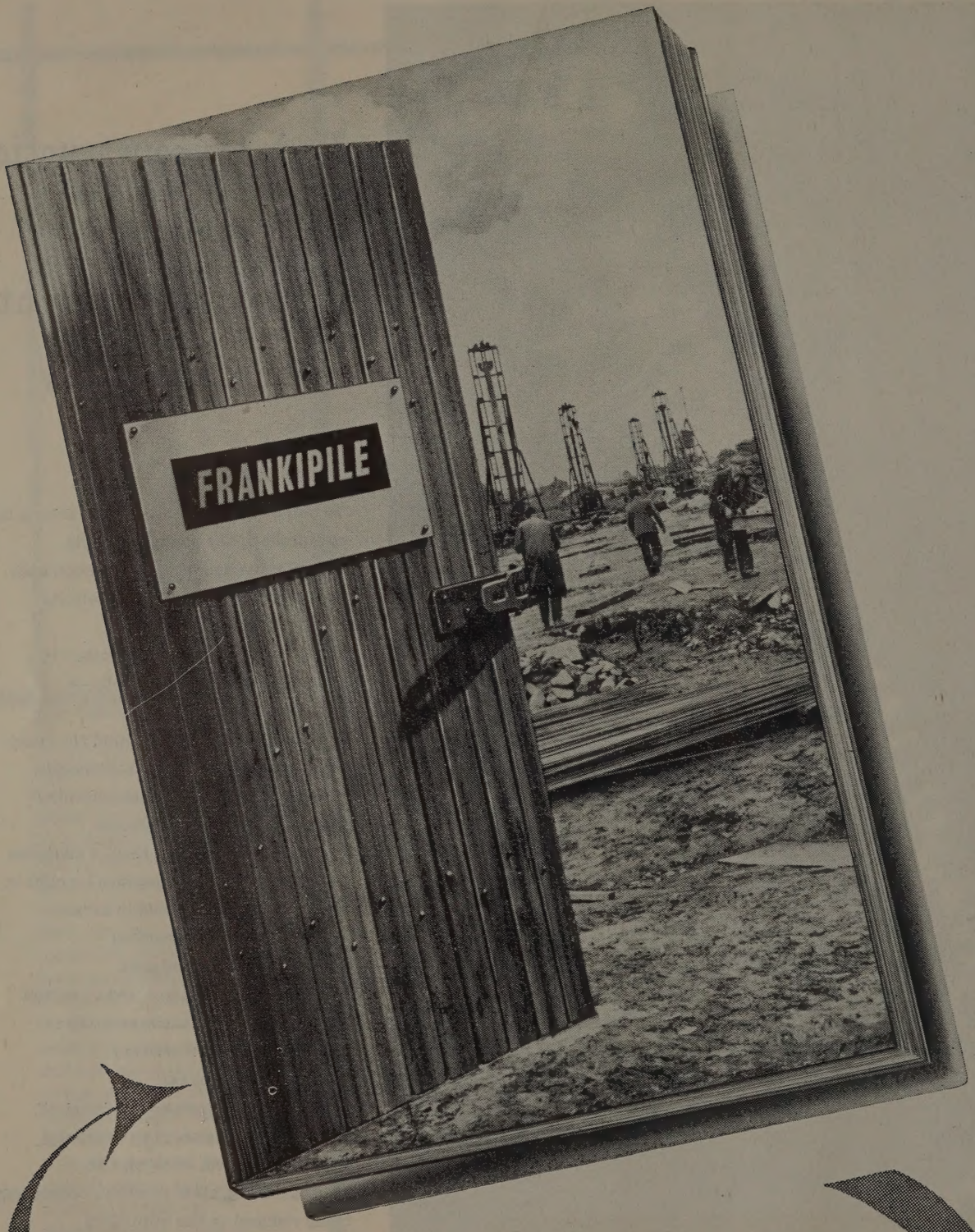

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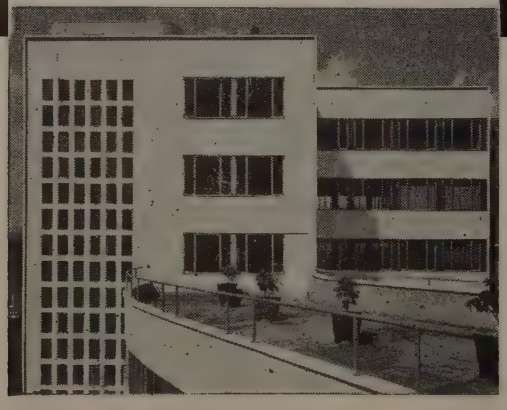

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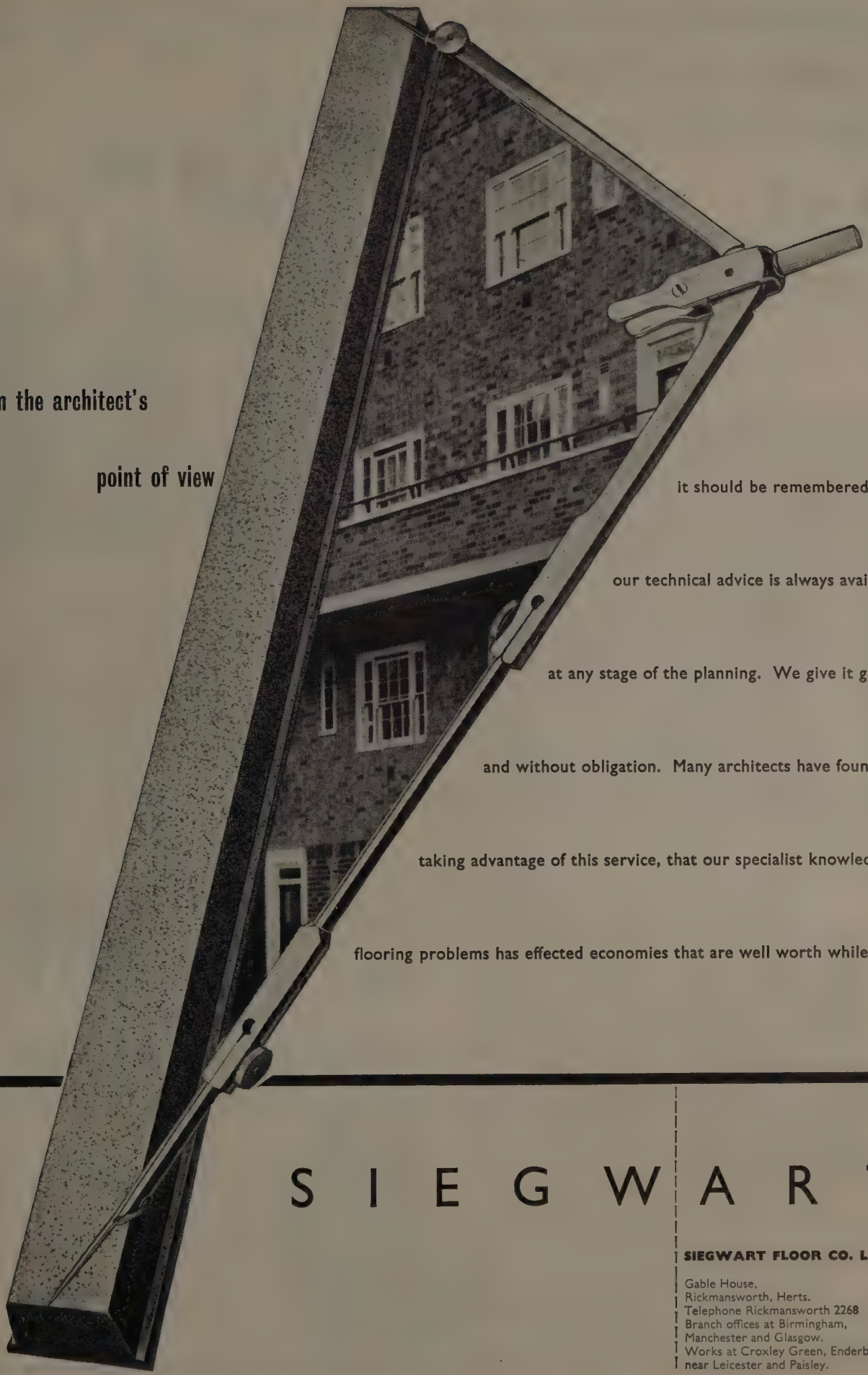

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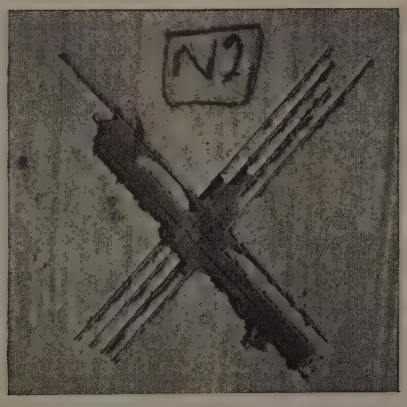

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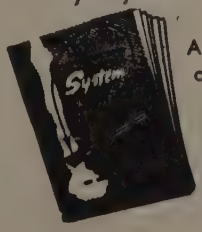

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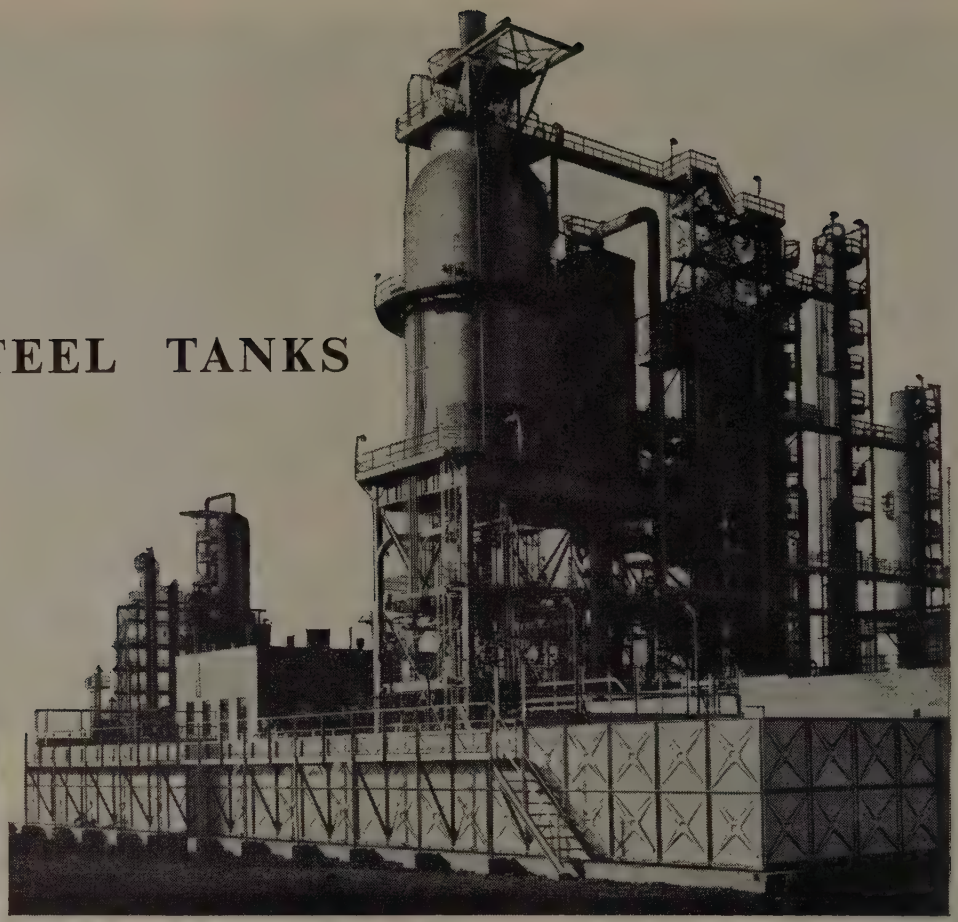

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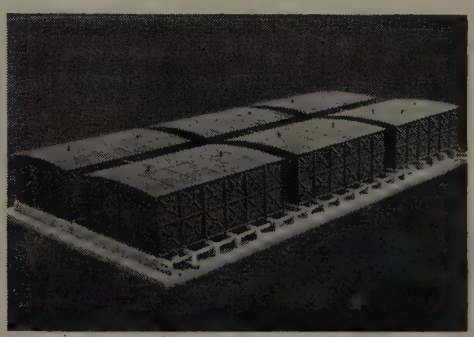
Pressed Steel Tank utilised as an architectural feature at a Secondary Modern School, Hunstanton, Norfolk, England.



A battery of tanks each containing 23,000 gallons at a frozen food preparation plant, West Lynn, Norfolk, England.

A solid fuel bunker which can be converted to store fuel oil, erected on the roof of a laundry, Slough, Bucks, England.

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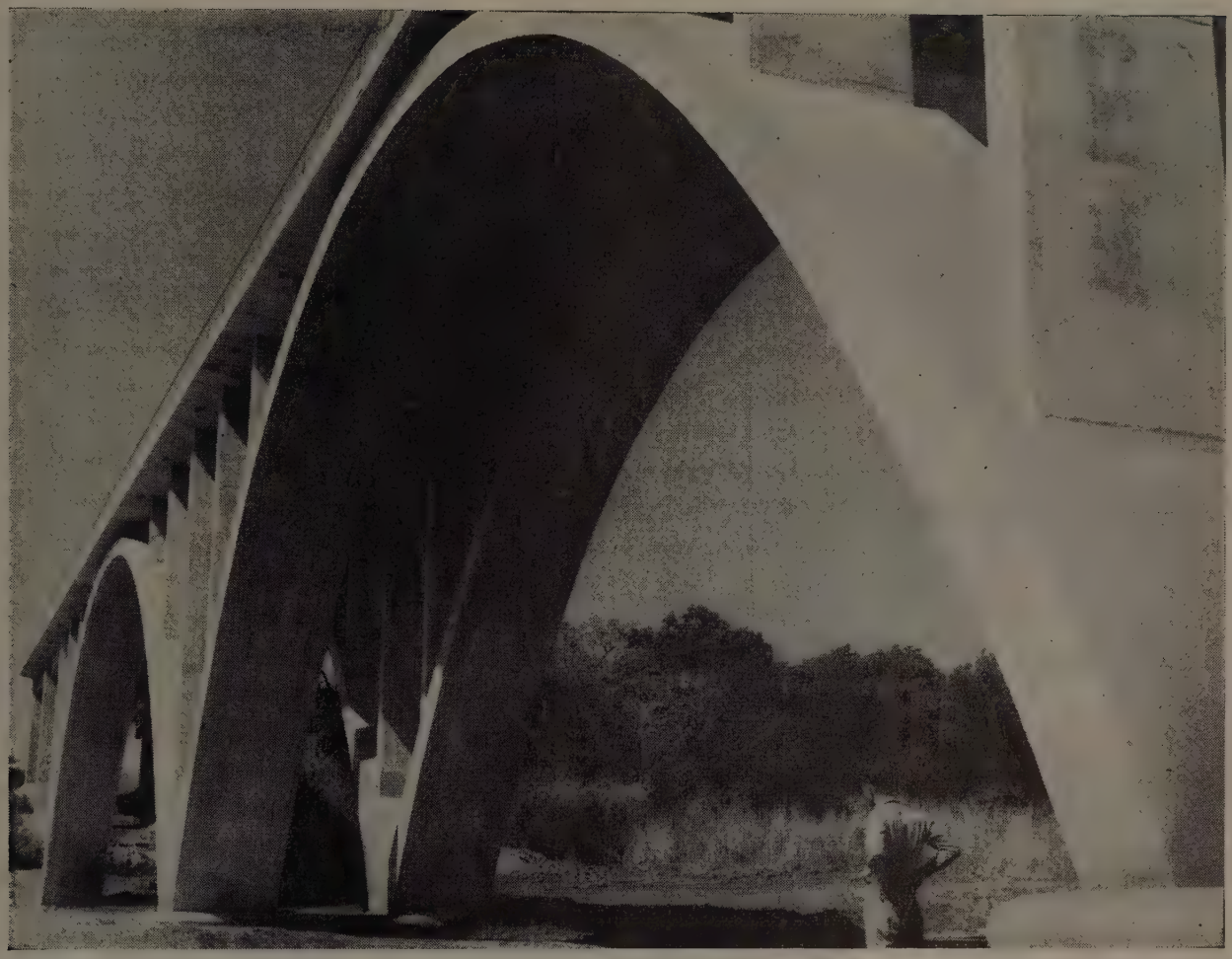

BRIDGE CONSTRUCTION

Reinforced concrete bridges in Portuguese East Africa, designed by the Public Works Department. Above, a completed bridge over the Revue River and below, a three-arch span bridge under construction over the Messica River.

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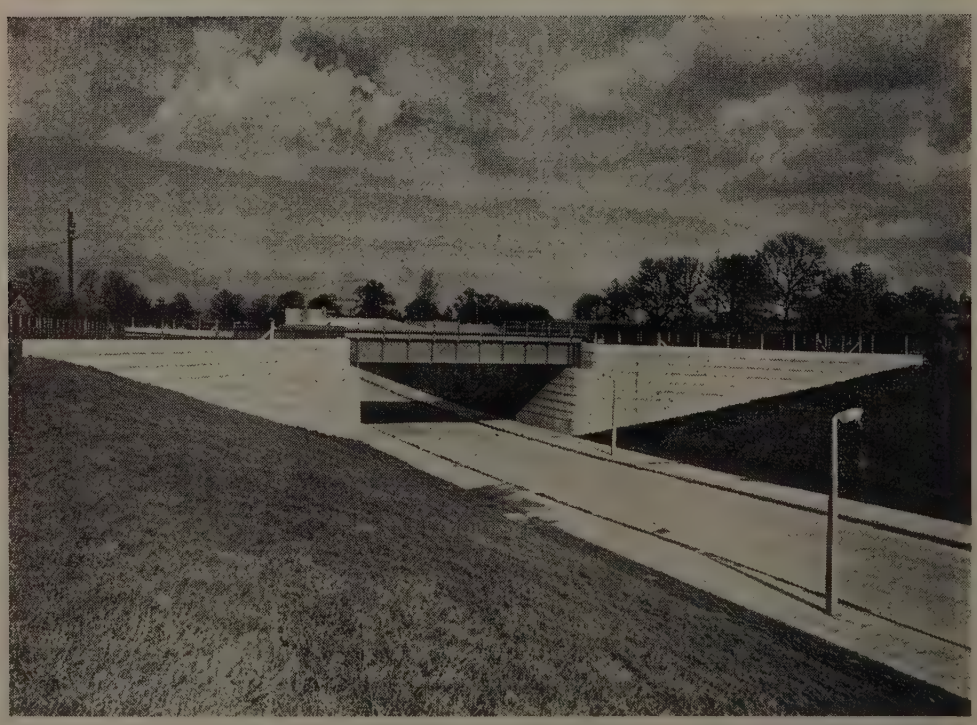

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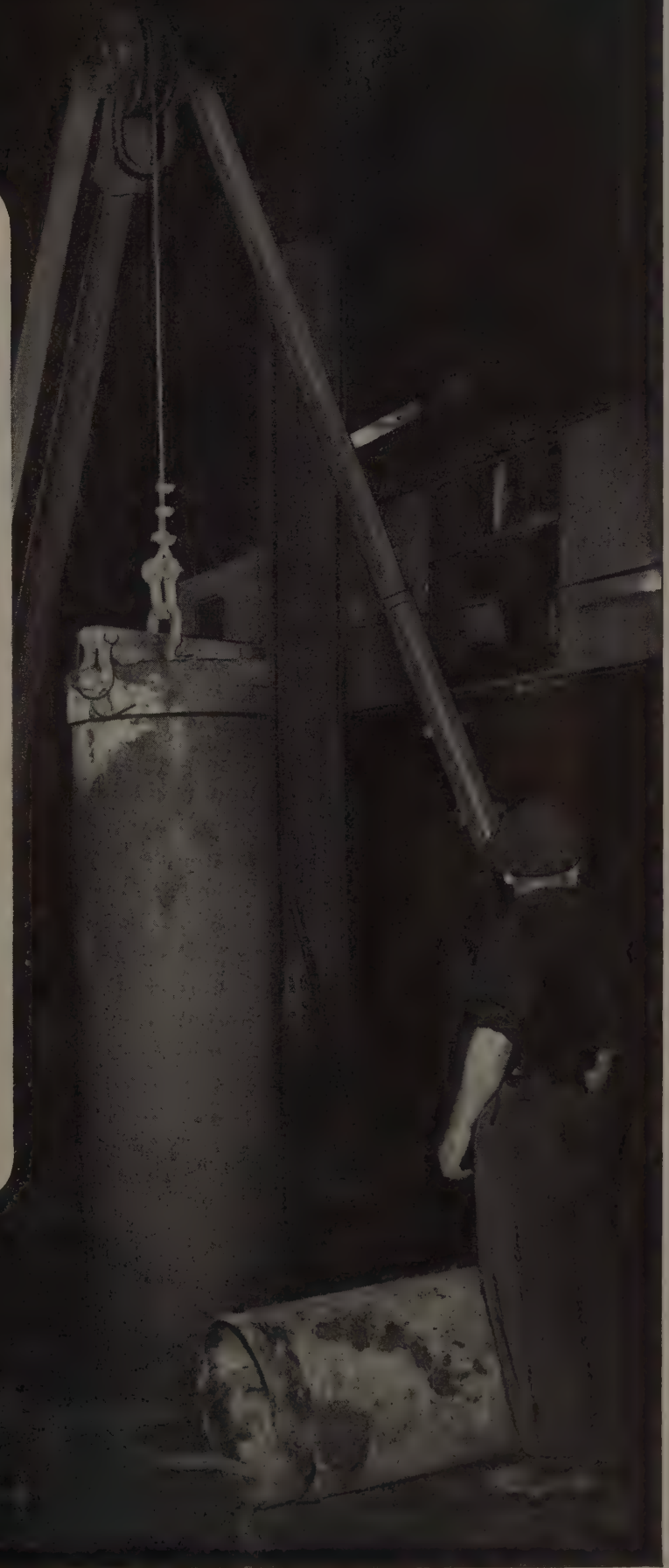

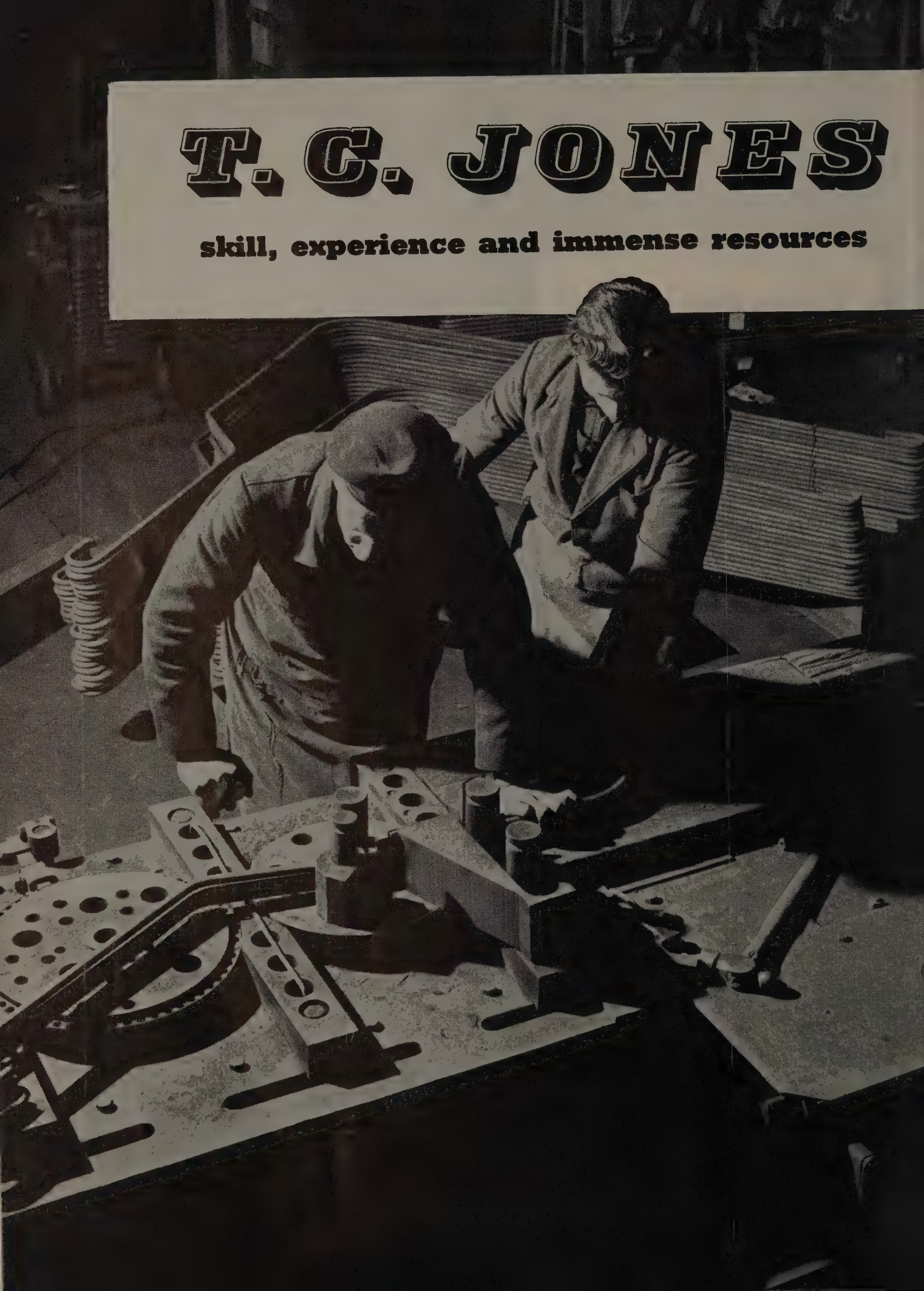
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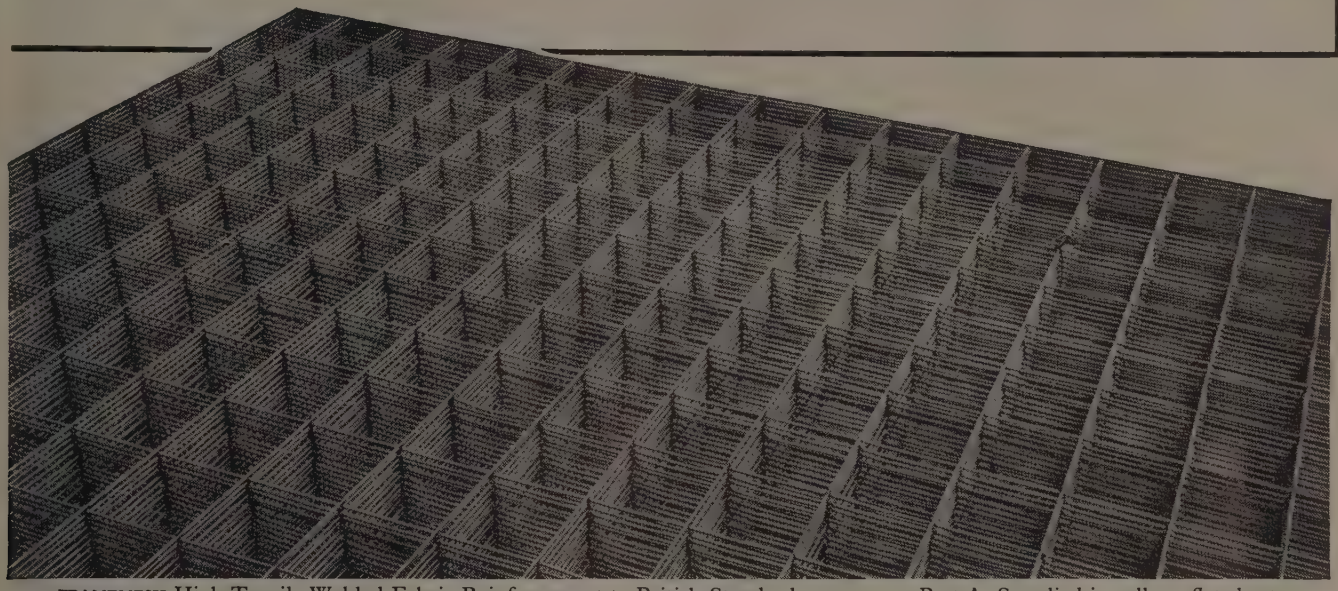

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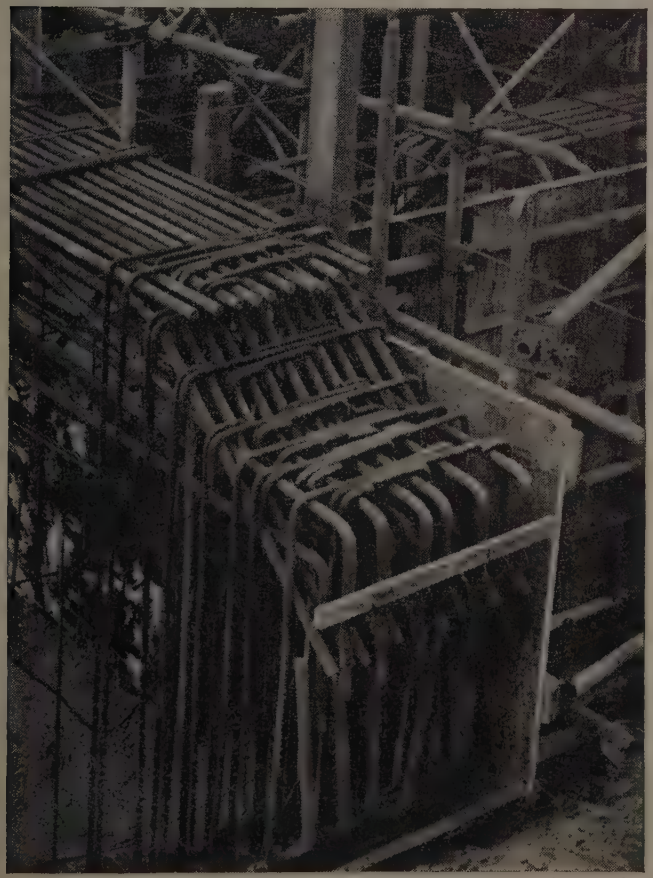

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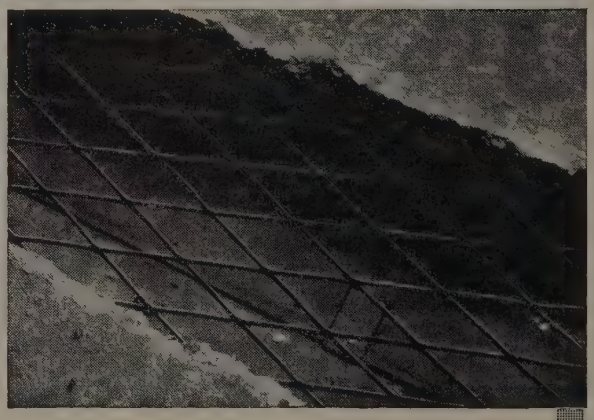

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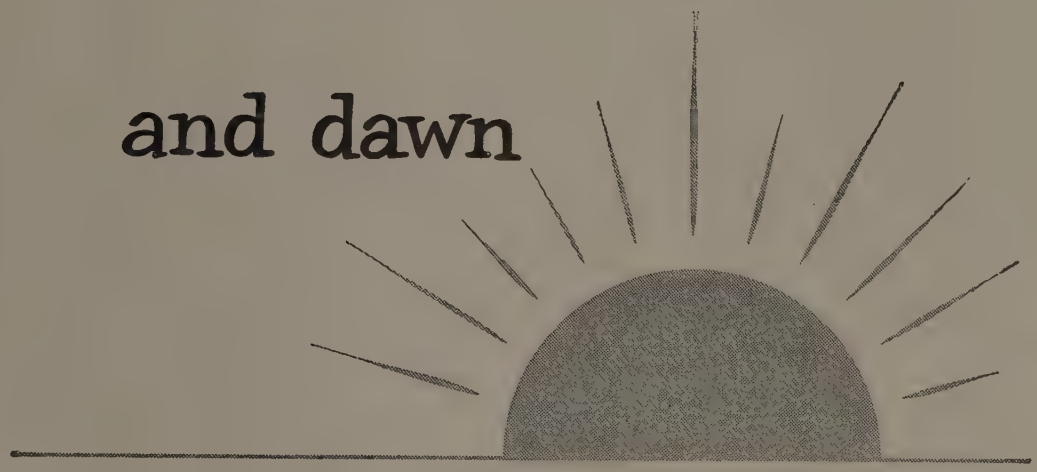

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THE STRUCTURAL ENGINEER

THE JOURNAL OF THE INSTITUTION OF STRUCTURAL ENGINEERS

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The Institution of Structural Engineers, whilst being at all times desirous of receiving communications from members and others, does not accept any responsibility for the opinions expressed by any individual Authors or Speakers.

Annual Subscription Rate to the Journal 54s. 0d. post free.

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The President, 1957-58

Prof. Sir Alfred Pugsley, O.B.E., D.Sc.(Eng.), F.R.S., M.I.Struct.E., M.I.C.E., F.R.Ae.S., whom the Institution welcomes as President for the Session 1957-1958, will take office on the 3rd October. As Professor of Civil Engineering in the University of Bristol since 1945 he is well known to a wide circle of members and many will also be familiar with his distinguished work in the field of structural engineering and aeronautical research.

Prof. Sir ALFRED PUGSLEY, O.B.E., D.Sc.(Eng.), F.R.S., M.I.Struct.E., M.I.C.E., F.R.Ae.S.

Alfred Grenvile Pugsley, who was one of six sons, was born at Wimbledon in 1903. His family was of North Devon origin. Educated at Rutlish School, Merton, Surrey, and London University, he graduated B.Sc.(Eng.), with First class Honours in 1923. He trained as a Civil Engineering Student Apprentice at the Royal Arsenal, Woolwich, under Lt.-Col. Hugh Mitchell, R.E., M.I.Struct.E., gaining experience in work on new office and factory buildings, river piers and jetties, roads and railways.

In 1926 he joined the Design and Research staff at the Royal Airship Works, Cardington, under Lt.-Col. V. C. Richmond, Chief Designer of the R.101. He took part in the design of various parts of the airship structure, particularly its main longitudinal girders, and acted as observer on the first full speed trials. In 1931 he was appointed to the scientific and technical staff at the Royal Aircraft Establishment, Farnborough, where he carried out research work on aeroplane structures, particularly in the new field now known as aeroelasticity. His work in connection with the airworthiness of particular aeroplanes involved full scale structural testing, including most of the wartime fighters and bombers. From 1941-1945 he was head of the Structural and Mechanical Engineering Department.

For his thesis on " Lateral Stability of Beams " he had been awarded the M.Sc.(Eng.) degree of London University in 1930 and this was followed by the D.Sc.(Eng.) degree in 1938. In 1944 he was awarded the O.B.E.

Dr. Pugsley was appointed Professor of Civil Engineering in the University of Bristol in 1945. He has conducted research work on structural problems of light alloy and cold rolled steel structures and suspension bridges and has worked on problems of structural safety. He took part in the planning of the new University Engineering Laboratories and was Dean of the Faculty of Engineering from 1951-1954.

In 1952 he was elected a Fellow of the Royal Society and in 1956 he received the honour of a knighthood.

Sir Alfred Pugsley has been associated with the Institution of Structural Engineers for over 30 years, having been elected a Graduate in 1924. He transferred to Associate-Membership in 1925 and to full Membership in 1944. He has taken an active part in the affairs of the Western Counties Branch, of which he was Chairman for the Session 1951-1952. Since 1952 he has been a Vice-President of the Institution and for a number of years he has served on the committees dealing with Finance, Membership, Bye-Laws, Science and Research and Awards. He is also Chairman of the ad hoc Committee on Structural Safety and a member of the ad hoc Committee on Aluminium Alloys.

He has contributed many papers to the Institution, the first " Some Problems in the Design of Steel Roof Truss Members " being given in 1927. This was followed by " Aircraft Structures " in 1941, " Some Experimental Work on Model Suspension Bridges "—which was read before the Western Counties Branch and for which he was awarded the Institution Research Medal in 1949—" A Simple Theory of Suspension Bridges " in 1953, and " Design for Safety and Efficiency " in 1957.

Besides his communications to the Institution of Structural Engineers, Sir Alfred Pugsley has contributed numerous reports and memoranda of the Aeronautical Research Council (of which he has been Chairman since 1952), papers in scientific journals and publications of the other engineering bodies, articles and reviews in the engineering press, and he has recently published a book on " The Theory of Suspension Bridges."

Since 1956 he has been a member of the Advisory Council on Scientific Policy.

The Institution is developing rapidly and the coming year will see the completion of its first half-century. It is a matter for congratulation that the direction of its affairs at this moment in its history will be in the hands of one who has done so much to further the aims and objects of the Institution and who, by his own

(*Continued on page 385*)

The Design and Construction of Certain Temporary Works, The Upper Silent Valley Reservoir, Co. Down*

By W. J. Shirley, M.I.Struct.E., A.M.I.C.E.

It is not intended at the present stage of the dam construction to anticipate a possible main paper on the permanent works, consequently this paper deals entirely with certain sections of the contractor's own temporary works and the construction under his control. The contractor's crushing and concreting plant operations will also be fully covered in any paper dealing with the construction of the dam.

THE structural engineering side of a civil engineering contractor's operations covers a very wide field, and the contractor's Head Office Design Department is required to handle structural problems in almost every type of structural material. The general procedure in a civil engineering contractor's organization would appear to be roughly as follows :

The Head Office staff design the major temporary works in broad principle at the tender stage, and during the execution of the contract they produce the detailed design. The Site Design staff adapt the given design to suit the existing site conditions, and therefore must be capable of advising the Head Office on the suitability of the suggested detailed design in relation to the site circumstances. The approach of the contractor's structural engineer to the design of temporary works is of necessity rather different from that of the consulting structural engineer to the design of permanent works.

Owing to the nature of permanent structures it must be assumed that the designer and contractor organize their thoughts and efforts to ensure the permanency of the structure, consequently the general approach to the problem is that once the design, in accordance with accepted standards, is completed then all concerned with the project will so organize their work as to achieve the specified finished structure. In general, therefore, all materials and labour are suited to the designer's standards, whereas with temporary structures for civil engineering works it is more often the case that the design is suited to the standards of the material and labour. The designer of these temporary works has certain advantages over the designer of permanent works in his approach to the problem. For instance, higher working stresses may be assumed, and the limits are governed by the designer's experience and not by Codes of Practice. Noticeable deflections and movements of the structure may be permissible. Appearance is not of great importance. Except in certain cases, the standards of accuracy of the work are more flexible than normal. If the structure is not suitable, alterations may be made without undue restriction, and a certain amount of trial and error may be assumed.

* *Paper read before the Northern Ireland Branch of the Institution of Structural Engineers at Belfast on the* 10*th January*, 1956.

The designer of temporary works also has certain disadvantages which govern his approach to the problem.

The choice of materials may be extremely limited, and in most cases the designer is dealing with materials of unknown specification, probably second-hand, and which may have already had many uses and abuses.

There are usually limitations, particularly in steelwork, to the sections and lengths available, and the structure has to be adapted to suit these members. There is a " specification of experience " in that the nature and possibilities of the workmanship are generally known—e.g., the possible standard of site welding. The structure is likely to be roughly handled, and subject to unforeseen loading. Speed of erection plays a very important part in the design, and there are many last-minute changes, even during construction. It is hoped that the works described in this paper, and their constructional details, will produce a picture of the structural problems and solutions which are the everyday task of the contractor's engineer. The works described represent the larger structures on this particular site. There are many smaller structures with their attendant design problems, including timber and steel bridges, retaining walls, and earthworks.

There are three separate structures to be described, namely :

(1) A large structural steel derrick gantry of 400 ft. span ;
(2) A timber and steelwork set of aggregate weigh-batching bins of 900 tons' capacity ;
(3) A reinforced concrete screening and aggregate storage bin, using precast concrete, of 1,800 tons' capacity.

The relative positions of these three structures are shown in the general site layout (Fig. 1).

Design and Construction of the Derrick Crane Gantry.

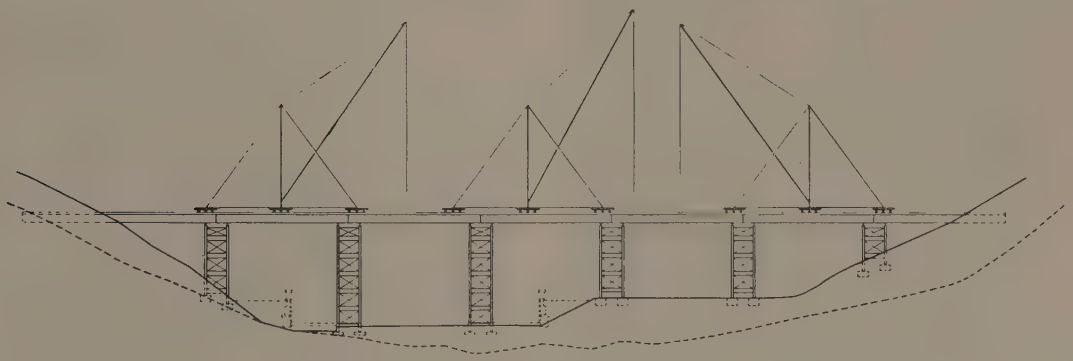
The cranage for the construction of the dam was based on the use of 10-ton electric derrick cranes, having 120 ft. long jibs and general capacities of 10 tons at 90 ft. radius and 4 tons at 116 ft. radius. The plan area of the dam to be constructed necessitated five of these cranes for complete coverage. The cross-section of the valley immediately downstream of the dam determined the siting of the cranes. This cross-section was such that the two outer cranes would have to be erected at a high level on the steep mountainside, and the three centre cranes were required to be travelling cranes, and in order to reach the top of the dam, the track level was to be not more than 80 ft. below the top of the dam. Therefore a crane track approximately 400 ft. long and 40 ft. above general ground level was required.

Fig. 1.—General Site Layout : Derrick Gantry, Weighbatching Bins, Screening and Storage Bins

For structural work of this nature the contractors had available a considerable quantity of Army ' V ' type steel trestle equipment, and certain 60 ft. long by 4 ft. deep plate girders, and the design was based on the use of this existing equipment, which had previously proved suitable for work of a similar nature.

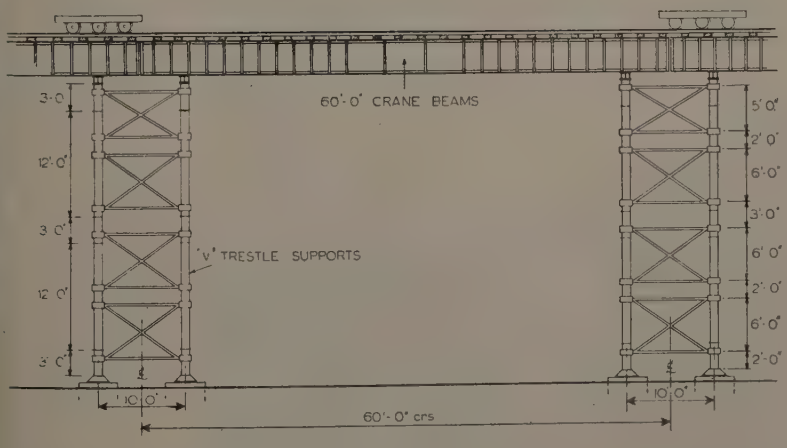
The general layout and dimensions of the Derrick Gantry are shown in Fig. 2.

The design loading on the gantry was that due to the dead and live loading from the derricks, the self weight of the structure, and the wind loading. The



ELEVATION OF DERRICK GANTRY.

Scale 1/16 in to 1ft.



ELEVATION OF ONE SPAN.

Scale 1/4 in to 1ft

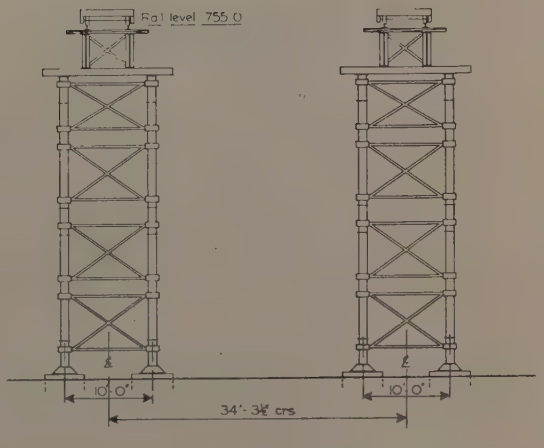


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Fig. 2.

derricks were erected with the king-post bogie on the upstream track, and the two ballast bogies on the downstream track. The dead loading for this type derrick, including all machinery and bogies, was assumed to be as follows :

King-Post Bogie	28 tons
Each Ballast Bogie	33 tons

(which last figure includes 26 tons of ballast, using 3 in. granite aggregate in boxes).

The live loading on these derricks was based on the 25 per cent overloading required for test purposes, i.e., 12½ tons lift at 90 ft. radius. It was assumed for design purposes that this live loading would allow for impact. This live loading gave the following bogie loads :

King-Post Bogie	46 tons
Each Ballast Bogie	23 tons

The design summary of dead and live derrick loading was :

King-Post Bogie	74 tons
Each Ballast Bogie	56 tons.

For wind loading, a maximum steady wind velocity of 75 m.p.h. was assumed for the site, and an equivalent wind pressure of 20 lbs. per sq. ft. of projected area was used for the wind calculations. In cases of shielding, as in the lattice construction, the area of one side was multiplied by a factor of 1:5. This wind loading was equivalent to a horizontal force of approximately 7 tons on the derrick, assuming that the jibs were down at maximum wind velocity.

The general design details of the main girders were as follows. The tracks were 4 ft. 8½ in. gauge, using 75 lb. flat-bottomed rail dogged to timber sleepers. Each alternate sleeper was bolted down to the top flange of the girders, and the bolting was staggered between front and rear girders. The king-post wheel-loading dead and live was 18½ tons per wheel, the wheel centres being 5 ft. 6 in. The ballast wheel-loading dead and live was 14 tons per wheel, the wheel centres being 8 ft. 9 in. The trestles were spaced at 60 ft. centres, and as the minimum distance possible between adjoining derrick king-posts was 75 ft., only one king-post loading was possible on each span. On the ballast road it was possible to have as a maximum condition on one span two ballast bogies 12 ft. apart due to adjacent derricks. The girders consisted generally of ½ in. web plates 48 in. deep, with top and bottom flanges fabricated out of two 6 in. x 6 in. x ⅝ in. angles, giving a 12½ in. wide flange.

The angles were riveted to the webs with ¾ in. rivets at 4 in. pitch. The web stiffeners were generally at 4 ft. centres, and were double 4 in. x 3½ in. x ⅜ in. angles. Certain girders differed from these typical girders in make-up sizes, and several had numerous existing unused bolt-holes, also some girders were in 30 ft. lengths with a centre site-bolted plated splice. Each girder was examined relative to the design details, and was erected according to its importance in the structure. The maximum bending moment due to the two rolling loads of 18½ tons on the king-post girders produced a tensile stress of 8 tons per sq. in., and the average shear stress in the web plate was 1½ tons per sq. in. Each girder was checked for stresses in the stiffeners, which varied considerably in layout, but with a maximum shear of 40 tons above stiffeners at 4 ft. spacing, all the stresses were low. For lateral buckling it was considered necessary to have the side supports at a maximum spacing of 16 ft., and for this purpose diagonal bracing between the two girders was provided at 12 ft. 6 in. centres, using 4 in. x 4 in. x ½ in. angles. This bracing was also designed to enable the two girders to act as a single braced girder in transmitting the horizontal wind loading to the trestles. The maximum deflection of the girders was calculated at ¾ in. The twin girders were supported on cill joists spanning across the tops of the trestle columns. These cill joists consisted of three 10 in. x 5 in. R.S.J.s with a ⅜ in. cover plate top and bottom, all welded together to form a compound girder.

These girders had a span of 9 ft., with two maximum point loads from the main beam reactions of 42 tons each. To avoid severe fixing stresses the main girders were only clipped to the cill joists with Lindapter clips. The cill joists were bolted to the tops of the columns, and the horizontal wind loading was transmitted to the trestles through these bolts.

The design of the trestle towers was as follows. The heights of the towers varied from 28 ft. minimum to 55 ft. maximum, and consisted of Army ' V ' type trestle equipment forming towers 10 ft. square. The loading on the trestle towers was resolved into three possible cases :

(1) Maximum vertical loading of 90 tons, with a horizontal load at the top of the tower of 9 tons.

(2) Vertical loading of 70 tons, with a horizontal load of 10 tons.

(3) Vertical loading of 5 tons, with a horizontal load of 6 tons.

These conditions produced a maximum load on a single column of 62 tons, and a calculated stress of 8 tons per sq. in.

Each trestle was designed as two single frames, and, although really redundant frames, they were checked as simple frames, with the diagonals taking tension only. No built-in stresses were considered, as each diagonal is adjustable to ¼ in. on the vernier.

The general details of Army ' V ' type trestling are as follows. The trestle equipment is constructed from normal mild structural steel, fabricated to very fine limits for absolute interchangability, and using electric welded construction.

The column units are in various lengths of from 1 ft. to 12 ft., and are fitted with end plates milled dead square and exact to length. The column shaft is $\frac{5}{16}$ in. plate 10¾ in. square, and is drilled accurately for the various connections. The column units are bolted together with ¾ in. black bolts, and when fully bolted the column strength in bending is developed. The longest unit (12 ft. long) weighs only 4 cwt. Base plates 3 ft. square and fabricated out of ½ in. plate are provided for the distribution of foundation loads.

All horizontal bracings are in three fixed lengths and of tubular construction, using 4 in. outside-diameter tubes ¼ in. thick, and are designed for loads of up to 15 tons.

As the columns are of very light construction, in order to transmit bracing loads of a high order, a method of attachment using a stout yoke is adopted. This yoke must be formed right round the column. The yokes are either plain or fitted with lugs for the bracing attachment. The bracing attachment is a single 1¼ in. diameter long machined pin. The yoke itself is bolted to the column shaft with ¾ in. black bolts. Yokes are connected to one another at corner junctions with 1¼ in. machined pins and 3 in. spacer washers.

Fig. 3.—Derrick Gantry. Top Section of Trestle, showing Cill Joists and Main Girders

General details of the ' V ' trestling, cill joists, and girder attachments are shown in Fig. 3.

The trestle bases were founded generally on the glacial drift, which consisted of sand and large granite boulders. They were taken to such depth as would ensure a safe bearing pressure of 3 tons per sq. ft. The bases were generally 5 ft. square by 2 ft. deep concrete pads with a reinforcement mat in the bottom. Certain of these footings were subsequently underpinned and deepened as excavation levels were increased. The underpinning was carried out by jacking up the trestle base, removing the base, and adding a further column length as required, and pouring a new base at the lower level.

The nature of the excavations and ground conditions adjoining the trestle foundations are shown in Fig. 4.

Considering the stability of the structure as a whole, the shorter trestles were calculated to have a factor of safety against overturning of 1.5, but in the longer centre trestles this factor was in theory reduced to 1.2 and it was considered desirable to brace the upstream and downstream trestles together. This bracing was constructed out of Bailey bridge sections, and designed to transmit horizontal loading equally to the trestles.

The central trestles with the Bailey bracing panels are shown in Fig. 5.

The excavation for the foundation bases was done by hand, and there were two methods of construction. Having obtained a suitable foundation for the base, a 3 in. concrete mat was immediately placed in the bottom. The reinforcement mat was placed on this concrete, and then (in the first case) the base was poured to the level of the underside of the steel trestle footing. The trestle was then erected on steel packings, and holes drilled in the concrete for the holding-down bolts. The bolts were fixed, the trestle set to level, and then the bolts and base were grouted.

In the second case, the trestle was erected over the foundation and placed to line and level resting on rails across the pit. The holding-down bolts were inserted, and the base poured complete to the underside of the baseplate.

In the early stages only a sufficient length of gantry was erected to enable a 10 ton derrick to be placed on the gantry. This first erection was done with a 5 ton derrick, which at the time was building the concreting plant in the same area. The trestles were made up in sections, lifted into position, and bolted together in position. When the first 10 ton derrick was in operation on the gantry it was then able to build the remaining sections of gantry as required, and to erect the other two 10 ton derricks.

The partially-constructed gantry with two derricks in operation is shown in Fig. 6.

The gantry was erected over a period of 12 months to suit the construction programme. The whole gantry

Fig. 4.—Derrick Gantry. Main Dam excavations adjacent to Trestle Footings

Fig. 5.—Derrick Gantry. Tall central Trestles with Bailey Panel Bracing

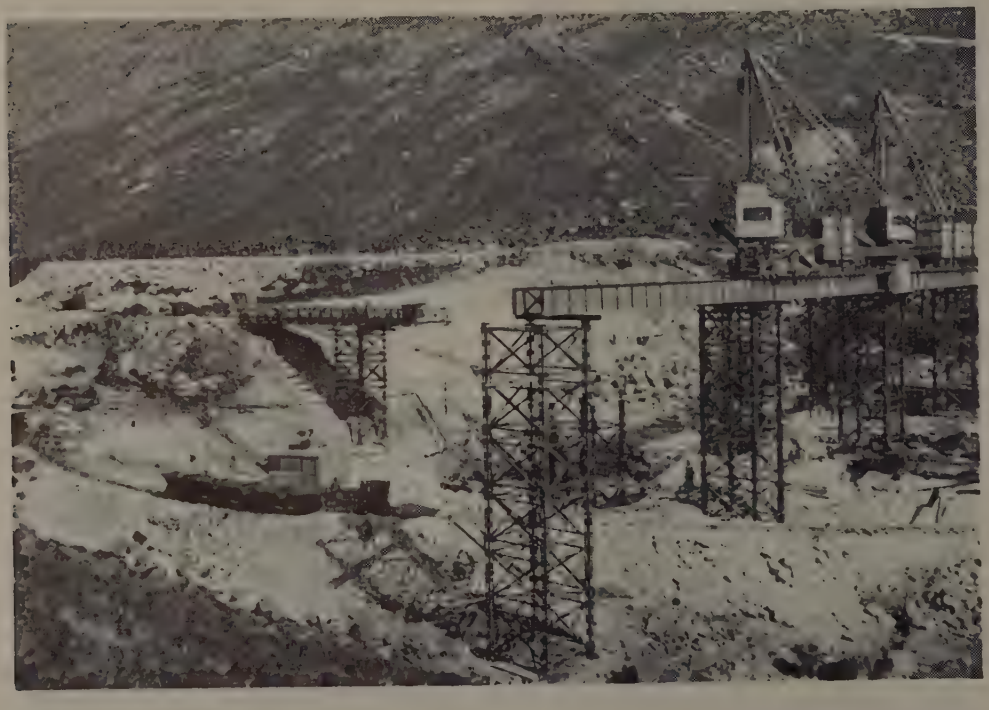


Fig. 6.—Derrick Gantry. Two-Derrick stage of Erection

was provided with a cantilevered walkway of tubular scaffolding construction. This walkway also carried certain air lines and water lines for the dam construction. The total tonnage of steelwork in the gantry was approximately 300 tons, and there were 200 cu. yds. of concrete in the foundations.

Fig. 7 shows the derrick gantry, cantilevered walkway and general top layout.

On the East Mountain hillside a 10 ton fixed derrick was erected, also using ' V ' trestle equipment for the supporting trestles. The isolated trestle under the king-post was required to be stiffened due to the slewing reaction from the derrick acting at the top of the trestle support. For this purpose a flying bracing of trestle construction was erected, tying the head of the king-post trestle to the ballast trestle foot.

On the West Mountain hillside it was necessary to provide a 10 ton travelling derrick at a higher level than the main gantry, in order to obtain complete coverage for this area of the dam construction. For this purpose another ' V ' trestle gantry was erected, of similar construction to the main gantry, with a portion of the derrick tracks cutting into the mountainside.

Fig. 8 shows five 10 ton derricks commanding the area of dam construction.

Fig. 7.—Derrick Gantry. Top Layout showing Cantilevered Walkway

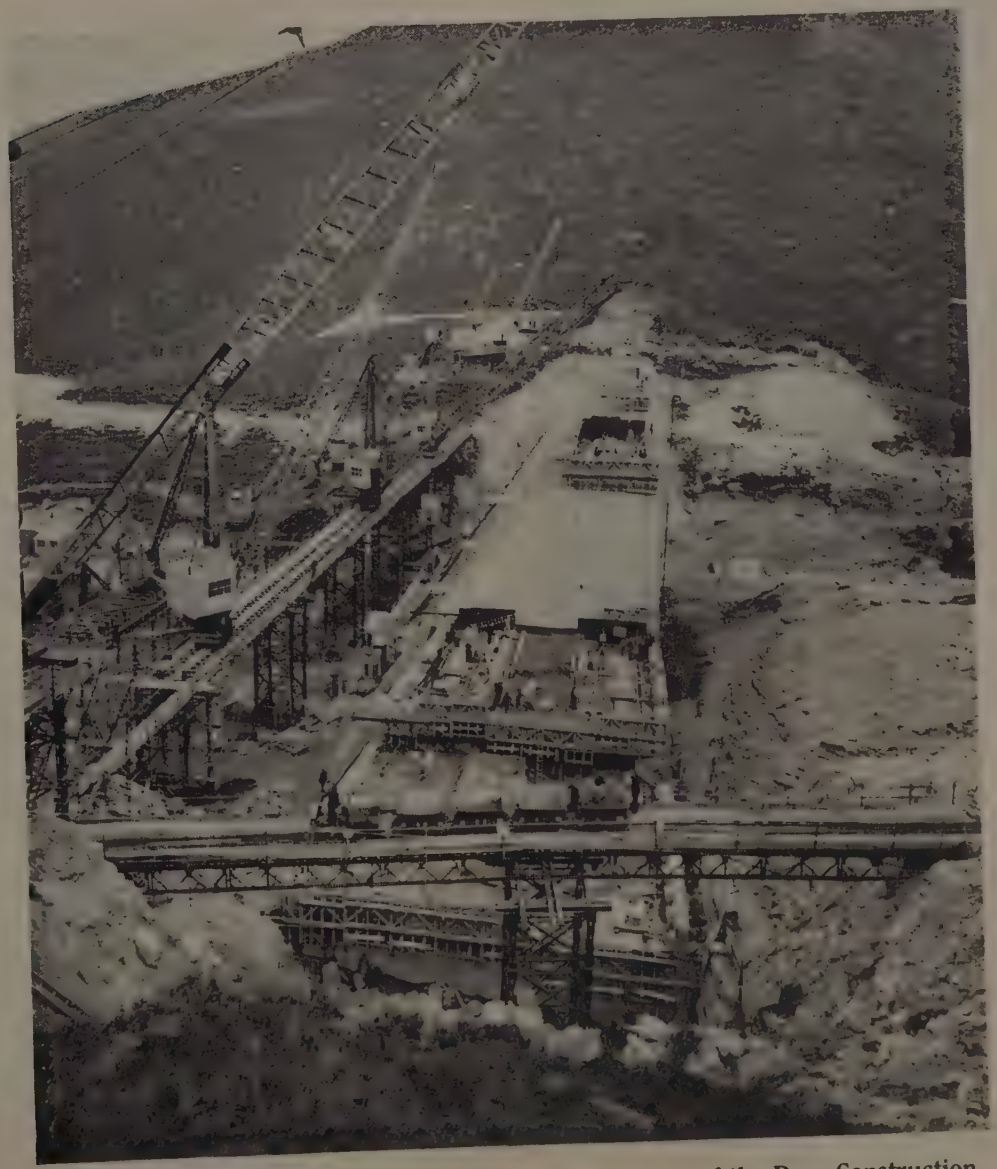


Fig. 8.—The Five 10-ton Derricks commanding the area of the Dam Construction

As a temporary measure it was necessary at one stage of the main dam rock excavation to run some of the excavated rock upstream of the dam, and in order to have a loading point within reach of the gantry derricks at a suitable level for the disposal tracks, it was necessary to erect a cantilevered two-track platform projecting over the downstream face of the excavations. A 'V' trestle tower was erected clear of the rock excavation, and a 'single single' Bailey bridge was supported on this tower, with one end cantilevered 25 ft. over the excavation. The other end was fixed to a timbered grillage on the high ground, and was counterbalanced with heavy boulders.

This cantilevered Bailey loading platform is shown in Fig. 9.

Fig. 9.—Cantilevered Bailey Loading Platform upstream of Dam Excavation

A high-level bridge was also constructed across the main dam excavation in order to carry a 2 ft. gauge track. This bridge was 100 ft. long, with a centre supporting trestle, and it was designed for a live loading of one 6 ton locomotive and six fully-loaded 2 cu. yd. rock waggons.

This bridge is of interest in that Bailey panels were used for the main beams, and all connections and cross-bracings were constructed in tubular scaffolding.

Fig. 10 shows the main access Bailey bridge with tubular scaffold bracing, and also indicates the adaptability of this type of trestle to footings at varying ground levels.

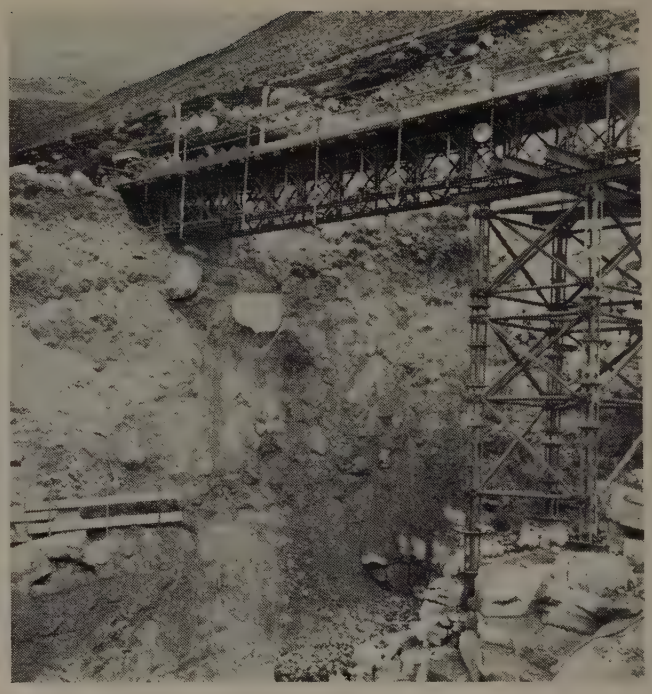


Fig. 10.—Main Upstream Access Bridge

The Design and Construction of the Aggregate Weighbatching Bins

The Problem. The requirements of the Specification for the concrete were such that it was necessary to store and weighbatch separately seven different sizes of concrete aggregates, i.e., five different stone sizes from 3 in. down to $\frac{3}{8}$ in., and two sands, natural and crushed. It was decided that the weighbatching bins for each of these sizes should hold 1 day's supply of aggregate for concreting. If, then, a breakdown occurred on any of the plant feeding the bins, there was sufficient aggregate to maintain the day's concreting programme. For this purpose it was necessary, therefore, to provide an average bin size of 130 tons capacity, i.e., a total weighbatching storage of 910 tons.

The site for the weighbatching bins was fixed by the general plant layout, and was on virgin ground previously excavated to a general level. The weighing mechanism and feed conveyors determined the heights and clearances of the structure.

It was decided to construct the bins using a structural steel frame and timber wall sheeting. The steel hopper bottoms were supplied with the weighing mechanisms. With the exception of the main stanchions, all the steelwork was available in the contractor's stock, consequently the design details were tied to the sizes and condition of the steelwork to be used. The stanchions were purchased new for the site.

The general layout and dimensions of the aggregate weighbatching bins are shown in Fig. 11.

The general design resolved itself into the design of a series of square bins of 20 ft. wall height, approximately 12 ft. square inside dimensions, with square hopper bottoms. The hopper bottoms to be supported 17 ft. above ground level. The superstructure on top of the bins consisted of a belt conveyor structure with a travelling belt tripper to feed the discharge into each bin as required, the whole conveyor system being provided with a walkway and a corrugated iron housing supported on a tubular scaffolding frame. The loading from this superstructure was fairly light, and was

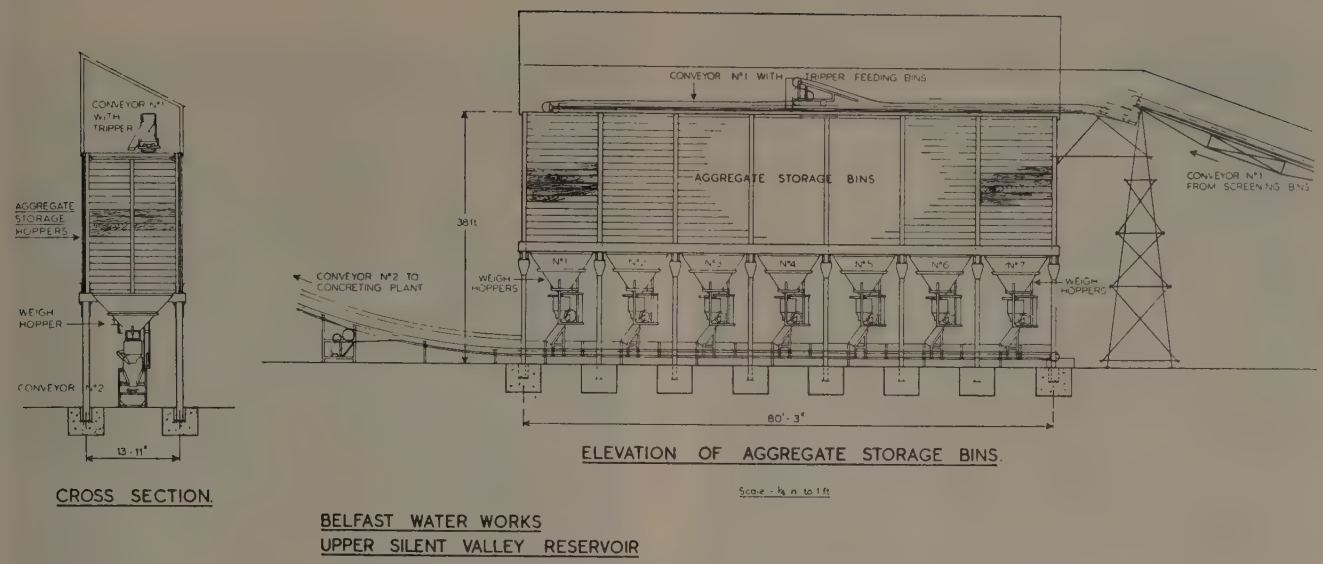


Fig. 11.—Aggregate Weighbatching Bins. General Layout and Dimensions

supported on the top cross wall ties, and consisted generally of a point loading of 1½ tons on each cross wall. The housing dead load was negligible, and the wind loading, taken at 15 lbs./sq. ft., produced a horizontal thrust of one ton on each cross strut.

The original design for the bin walls was based on the use of 3 in. timber sheeting of 6 ft. span, with intermediate vertical steel members.

These intermediate vertical members were required to transmit their horizontal loading on to the top and bottom side and cross beams. This design, although rather complicated, showed a considerable material saving, particularly in the timber sheeting; but the amount of site steel fabrication required for the side and cross beams which were required to take loading in two directions plus direct tension stresses was considered as likely to throw the steel erector's work out of balance in relation to the other site works, and possibly cause an erection delay. It was therefore decided to simplify the structure considerably, and to use more timber by providing only the four corner stanchions to take all the side loading, and to span the timber wall sheeting over the 12 ft. span. The wall loading calculations were based on the Janssen formula, the maximum loads occurring in the sand bins. It was assumed that the walls did not carry any vertical loading from the contained materials. The maximum horizontal loading at the bottom of the walls was taken as 520 lbs./sq. ft., and the maximum allowable stress in the timbers as 2,000 lbs./sq. in. The timbers commenced at 6 in. thickness, and were reduced to 4½ in. and 3 in. thick in accordance with the horizontal pressure diagram. The side wall timbers were cut to length and wedged tight between the stanchion flanges. The cross timbers were held in vertical channels formed by welding two angles on to the stanchion flanges.

The bin bottoms were assumed to carry the full load of the contained material, and were designed as square hopper bottoms in $\frac{5}{16}$ in. mild steel plate, with angle strengthening. These hopper bottoms were bolted to the side and cross beams, with angle connections. The loading from the hopper bottoms was assumed to be equally distributed on the side and cross girders at the top of the hopper level.

The Top Wall Frame. The top cross wall ties consisted of 10 in. x 5 in. R.S.J.s bolted and welded to the top of the stanchions, the joint being capable of a 10 ton tension. In general, all connections of tension members to the stanchions were designed on the basis that the maximum number of bolts were provided, limited only by available bolt room, and the balance of the tension taken on welding. The longitudinal ties were 6 in. x 6 in. x $\frac{3}{8}$ in. angles, bolted and welded into a continuous length, and designed for a 5 ton tension. The housing sheeting was fixed to these ties.

The Bottom Wall Frame. The hopper bottom supports across the bins were double 15 in. x 5 in. R.S.J.s, bolted and strapped together, and supported on each side of the stanchions on a plated bracket. These cross beams were designed for a vertical loading of 3½ tons/lin. ft., and a direction tension of 15 tons.

The longitudinal side bottom beams were also double 15 in. x 5 in. R.S.J.s bolted together, and designed for a vertical loading of 4 tons/lin. ft., and a direct tension of 6 tons. These beams were supported on the cross beams and their loading transferred to the brackets. The cross and longitudinal beams were bolted to the brackets, and the balance of the tension taken on welding.

The Main Brackets. The loading on these brackets was a maximum of 87 tons, and it was assumed that two-thirds of the load was taken on the inner bracket and one-third on the outer bracket. The internal brackets were 1 in. plate, and the external bracket $\frac{3}{4}$ in. plate. The brackets were held with 1⅛ in. bolts, full loading being taken on the bolts, maximum shear 5½ tons/sq. in. There was a certain amount of tack welding, but this was not assumed to be load-carrying. The condition of one bin full and the adjoining bin empty produced a case of 50 tons eccentric loading on one side of the bracket and column.

Main stanchions. These were required to carry the full vertical loading of the structure, plus the bending in two directions due to the horizontal loading of the contained material. The vertical loading on the stanchion was 90 tons, and the sections were 12 in. x 6 in. x 65 lbs. R.S.J.s, 40 ft. long. The design of these stanchions was rather involved, and originally they were assumed to be pinned at the top, fixed at the bracket level, and pinned at the base; but on final design a certain amount of fixity was assumed at the base. The maximum compressive stress was 10 tons/sq. in., and the tensile stress 6 tons/sq. in.

The end corner stanchions were required to carry more severe unbalanced loading, and it was decided to introduce an intermediate vertical member in the end walls to transmit a proportion of the horizontal loading to the top and bottom column connections direct. It was necessary to stiffen the stanchion section in its weak direction by the addition of a 4 in. x 3 in. R.S.J. section welded to the web, and, in addition, to reduce the bending moment at the bracket fixity, a tie was introduced below the bracket and a strut above the bracket. Each stanchion had a $1\frac{1}{4}$ in. base plate welded to the bottom, and the stanchions were placed on steel packings in a deep pocket formed in the base, and held in position by side timber wedges. The stanchions were then concreted in position.

Foundations. These consisted of plain concrete bases 5 ft. 3 in. square, 4 ft. 3 in. deep, with 17 in. x 20 in. x 25 in. deep pockets to receive the stanchions. The stanchions were slightly offset from the centre of the base as seen in the cross section to obtain an even load distribution. The foundations were designed for a bearing pressure of $3\frac{1}{2}$ tons/sq. ft. The structure when erected was sheeted with corrugated iron up to hopper level on two sides only. The bins were filled as evenly as possible on initial loading.

Construction. The stanchion bases were hand excavated, and concreted immediately each base was bottomed up.

The stanchions were erected in the pockets in the bases, using a 19.R.B. crane, and were held on guys until timber collars had been erected at gusset level, then each set of four stanchions was bolted together with timber frames. The four stanchions were then plumbed and concreted in the pockets. The brackets were marked off on the stanchions, the stanchions drilled, and the brackets fixed. The side and cross beams then followed, fixed by site drilling and bolting.

The structural steel framework of the aggregate weighbatching bins is shown in Fig. 12.

The top tie frames were then fixed, and when the main frame was completed, the welding was commenced. The side wall sheeting followed after the welding, and the cross wall sheeting was omitted until the hopper bottoms were fixed. The bottoms were lowered down inside the frame, and the supporting beams marked off for drilling. The hoppers were then lifted on block and tackle, and relowered and bolted when the drilling was completed.

The arrangement of the hopper bottoms and supporting steelwork is shown in Fig. 13.

The cross timber walling then followed as each hopper was erected. The superstructure and sheeting were erected after the plant installation and testing.

The superstructure housing and feed conveyor are shown in Fig. 14.

There were 80 cu. yds. of concrete in the bases and main operating floor at ground level. The weight of structural steelwork used was 35 tons, and the timber sheeting amounted to 2,000 cu. ft. The main frame and sheeting, including the plant erection, was carried out in sixteen weeks during the winter.

In the early stages of operation it was found necessary to provide felt sheeting to the external walls of the two sand bins in order to prevent driving rain penetrating through the timber walling and causing variations in the moisture content of the sands.

The finished structure is shown in Figs. 15 and 16.

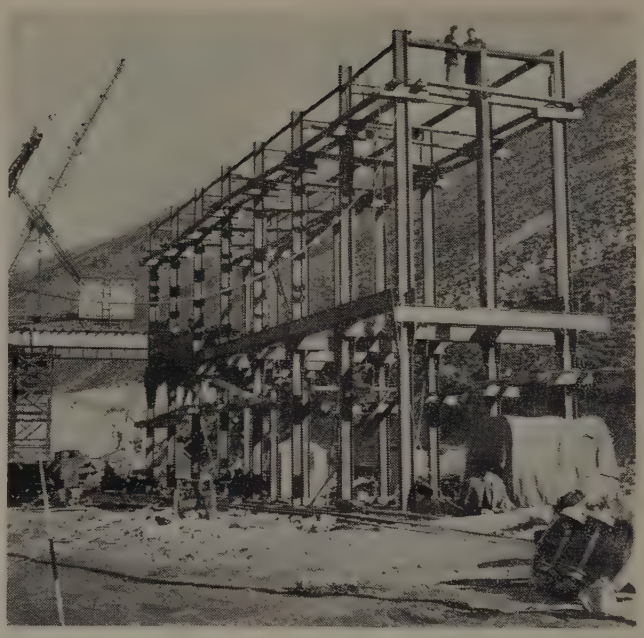


Fig. 12.—Aggregate Weighbatching Bins. Structural Steel Framework during Erection

Fig. 13.—Aggregate Weighbatching Bins. Arrangement of Hopper Bottom supporting Steelwork

Fig. 14.—Aggregate Weighbatching Bins. Feed Conveyor Housing and Steelwork to top of Bins

Fig. 15.—Aggregate Weighbatching Bins. Side Elevation

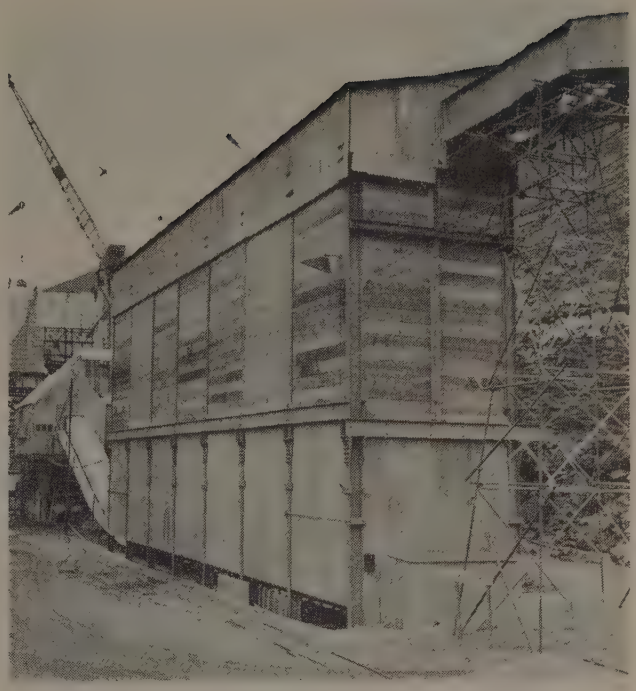


Fig. 16.—Aggregate Weighbatching Bins. View of Completed Structure

The Design and Construction of the Screening and Aggregate Storage Bins.

The Problem. The general decision on the layout of the crushing, handling and storage of the concrete aggregates was that from the time of entering the primary crusher all materials were to be under cover until they emerged from the mixers. For this purpose it was necessary to provide storage bins under the screens. The size of these bins was determined by the rough economics of the problem, and eventually it was decided to store sufficient materials for three days' supply. This required a total capacity of 1,800 tons, and an individual bin capacity of 300 tons.

Six bins were required for the different size screenings. The site of the screen house and bins was determined by the general plant layout, and it was to be partly on a rugged hillside and partly on a general levelled area. This levelled area was formed in the main by filling, and the storage bins had to be moved further into the hillside than anticipated in order to ensure that the foundations were on the virgin ground.

The general layout and dimensions of the storage bins are shown in Fig. 17.

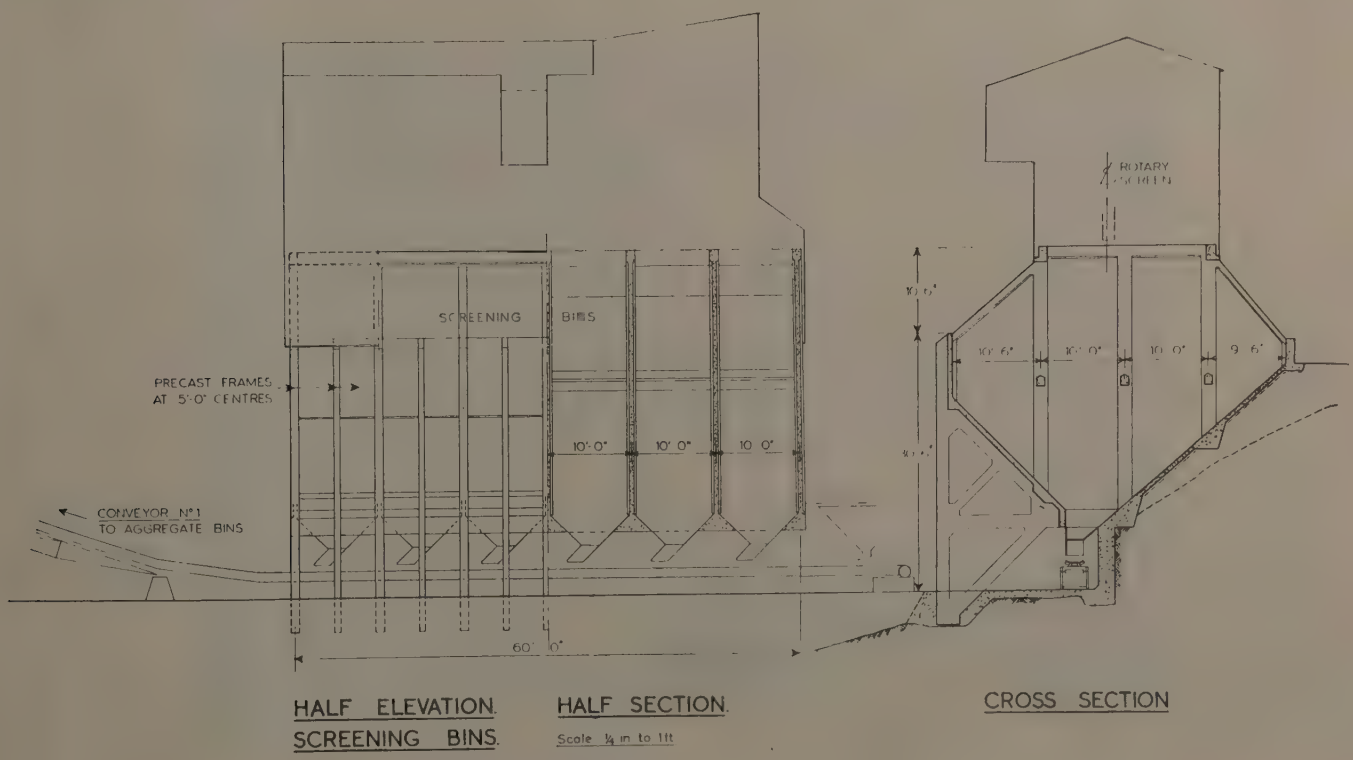


Fig. 17.—Screening and Storage Bins. General Layout and Dimensions

Main dimensions. The crushing and screening layout determined the top bin level, and the discharge and feed arrangements to the day bins fixed the hopper outlet level. These limits provided only 34 ft. height for the bin capacity, and any endeavour to increase this height only made the crushing layout more difficult. The screening arrangements limited the length of the bins in relation to the height, and the maximum distance which could be allowed between the bin partition walls was 10 ft.

The problem, therefore, was to obtain the capacity required by making the cross-sectional width as large as possible. This maximum width was obtained by making the hopper side slopes as long as possible, and, in effect, there was practically no vertical side walling.

The side slopes were 40° minimum, and the top cross-section of the bins was shaped to the approximate angle of repose of the stored material. Having decided to move the structure into the hillside, it was then apparent that the hillside would be utilised to form one of the sloping sides of the bins.

Materials. The general dimensions and shape of the bins having been roughly determined, the question then arose regarding the nature of the materials of construction. The unusual shape of the structure precluded any attempt at a structural-steel-and-plated bin which could be considered as having future uses on other contracts. A composite structure of steel and timber might have been economical if the walling were vertical, as in the day bins, but it was considered that all the load-carrying sloping wall sides would have to be of reinforced concrete. It was finally decided, in view of the other steel and timber structures under erection at the same time, that the structure should be in reinforced concrete. Also, the contractor had readily available at an economic price a sufficient quantity of suitable reinforcement which was surplus to requirements on a previous contract. Stone for aggregates was readily available on site, and use was made of a small portable crusher for producing these. A 10 ton derrick commanded the site of the bins, this being necessary for the erection of the crushers. The choice of the reinforced concrete structure allowed a better distribution of the labour force, and assisted with the build-up for the main job of a good carpenter gang and concreting gang. It was decided to use precast concrete as much as possible.

The Design. The general design was based on constructing an in-situ concrete slab and retaining wall on the hillside to form one slope, with precast concrete frames and in-situ slabs forming the other side slope. The division and end walls were to be precast, with in-situ vertical wall beams and horizontal ties framing the whole structure and supporting the screening machinery and screen housing. The internal division walls were designed as 6 in. reinforced slabs spanning horizontally between the vertical wall beams. The slabs were designed for an unbalanced pressure of one bin full and the adjoining one half-full. They were designed as fully continuous, and in addition to the bending stresses the vertical wall slabs were required to take the direct tension stresses due to the side wall pressures. The main slab reinforcement was $\frac{1}{2}$ in. bars at 8 in. centres in both faces with distribution steel of $\frac{3}{8}$ in. bars at 12 in. centres. The end vertical walls were required to take the load of one bin full, and were 7 in. thick slabs, with $\frac{1}{2}$ in. bars at 5 in. centres on the tension face. The maximum loading on the walls was taken at 580 lbs./sq. ft.

The vertical wall beams were supported in the middle by the horizontal tie beams, and were designed over two spans, i.e., from bottom of the hopper to the centre tie and from the centre tie to the top wall tie.

These columns or beams were 12 in. thick and 21 in. wide, and had six $1\frac{1}{8}$ in. bars in the bottom span, and four $1\frac{1}{8}$ in. bars in the top span. The top tie beams were on the outside vertical beams only, the centre top tie beam was omitted to suit the plant layout, therefore the top of the division wall was designed as a horizontal beam to take the centre vertical beam reaction. This beam spanned between the outside tie beams. The top outside tie beams were 24 in. deep x 12 in. wide, and were designed to take the reactions from the columns in tension and the weight of the superstructure. The loading from the screening machinery was carried on the vertical cross walls. The centre tie beams were tension members only, and were 15 in. x 12 in, with four No. $\frac{7}{8}$ in. bars to take a tension of 16 tons. The bottom tie beams to the vertical beams were formed in the sloping slabs, and also formed the supports to these vertical wall beams. The side sloping wall formed on the hillside consisted of a 6 in. cast in-situ slab poured against a preformed slope. This slope was made up with boulders and weak concrete to the correct slope. Only nominal reinforcement was placed in this slab, just sufficient to transmit the sideloading from the cross walls for the stability of the structure. The outside sloping walls were designed as in-situ slabs spanning between frame supports at 5 ft. centres. These slabs were designed as continuous spans for a maximum loading of 1,900 lb./sq. ft., plus direct tension stresses. The slabs were tied into the frames and cross walls.

A retaining wall 8 ft. high was formed at the toe of the hillside slope in order to give clearance for the hopper discharge, and to retain the hillside and act as a toe for the side walls and a foundation for the main structure. This wall was 2 ft. thick, and was reinforced with $\frac{7}{8}$ in. bars at 9 in. centres. The hopper bottoms were formed against this wall, and the external precast frames were tied to it. The outside sloping walls and one half of the bin structure and contained materials were supported on thirteen reinforced concrete frames at 5 ft. centres. There was one frame at each cross wall, and an intermediate frame between the walls. The intermediate frames were designed to carry an inclined load of 90 tons, and the frames at the cross walls were designed to carry a load of 120 tons. The larger frames were 12 in. thick by 18 in. cross-section, and weighed 9 tons, and the intermediate frames were 9 in. thick and weighed 7 tons. The reinforcement was generally four No. 1 in. diameter bars with $\frac{5}{16}$ in. links at 9 in. centres. The foundation for these frames was designed as a continuous footing for a total load of 1,400 tons. This was a reinforced concrete beam foundation, 65 ft. long, 6 ft. 6 in. wide, and 3 ft. 3 in. deep, with pockets at 5 ft. centres to receive the frames. The ground bearing pressure was $3\frac{1}{2}$ tons per sq. ft.

At the downstream end of the bins, the hillside sloping wall was extended some 15 ft. to form an apron chute, and natural sand was tipped by lorries down this chute on to an extension of the conveyor which received the stone from the storage bins.

Construction. The first operation was the construction of the hillside retaining wall. This was carried out in very wet and bouldered ground, and weep pipes were left in the wall to drain the slope and avoid any build-up of pressure. Having constructed the retain-

Fig. 18.—Screening and Storage Bins. Lifting precast Frame off Casting Bed

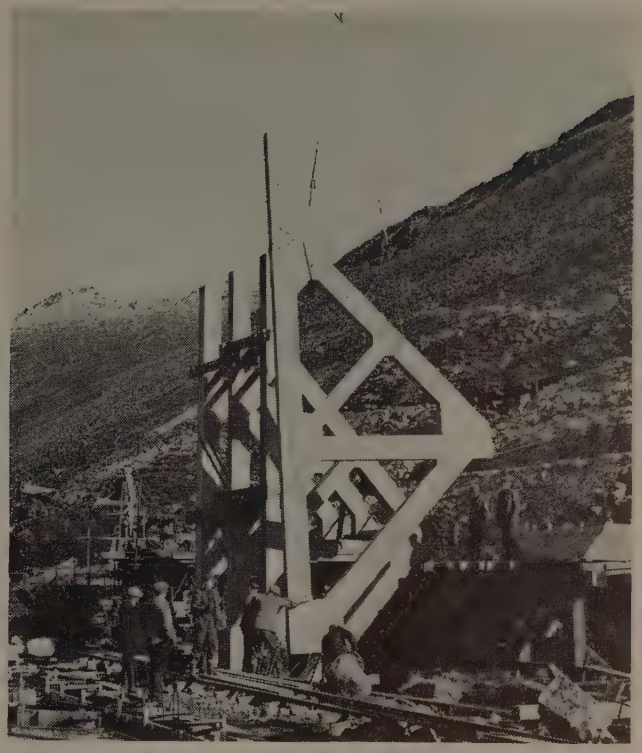


Fig. 19.—Screening and Storage Bins. Setting precast Frame in position

ing wall, the boulder slope was formed, handling the boulders with the 10 ton derrick.

The precast frames were cast flat on a ground slab within reach of the derrick. They were lifted at 4 days, and erected at 7 days.

Figs. 18 and 19 show the lifting and setting into position of the precast concrete frames.

The frames were erected in pockets in the foundation beam, with the hopper end of each frame supported on a timber trestle. The frames were bolted together with timber collars during erection, and, when lined and levelled, the base was concreted into the pocket. The construction of the hopper bottoms was the next operation, the hopper mouth beams forming ties between the

Fig. 20.—Screening and Storage Bins. Erection of precast Wall Slabs

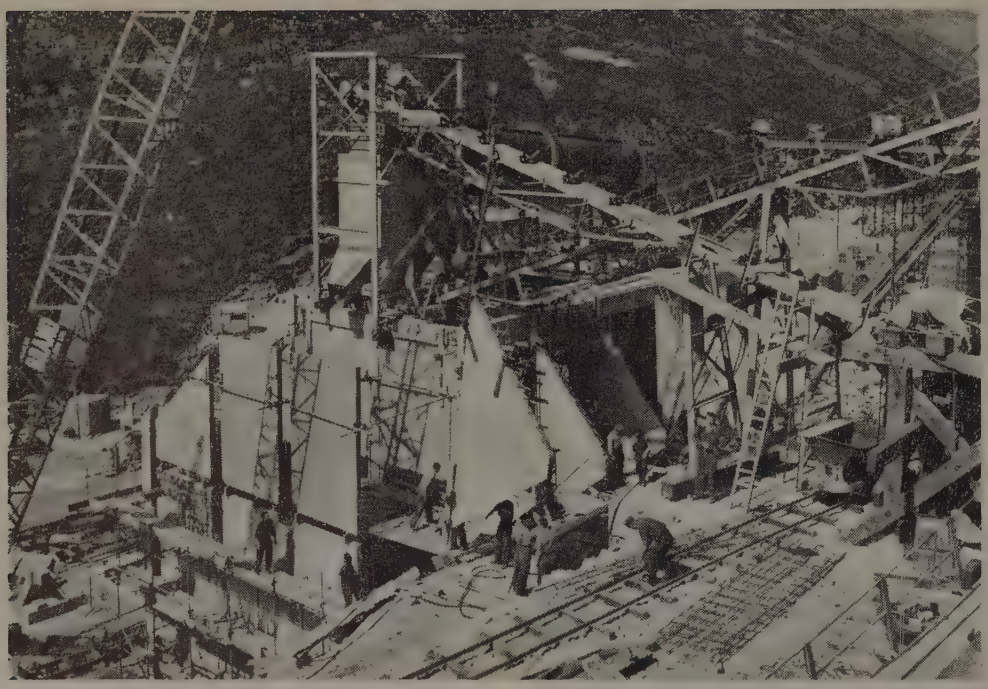


Fig. 21.—Screening and Storage Bins. Erection of End Wall Slabs and Screening Plant

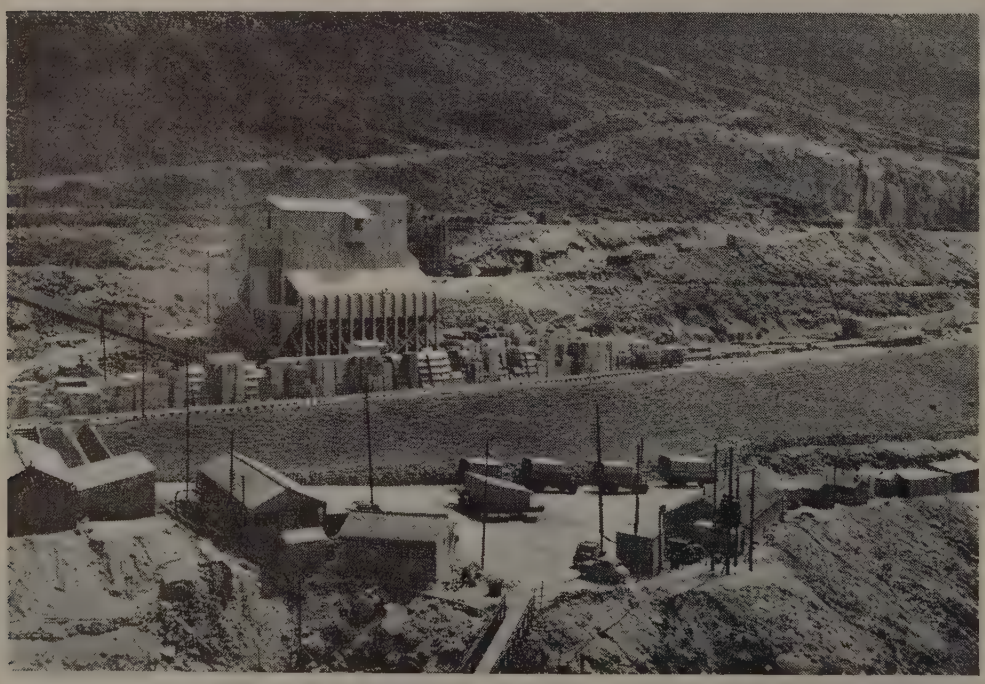


Fig. 22.—Screening and Storage Bins. General View of Completed Structure

precast frames and the retaining wall. Having constructed the hopper bottom, the frames were then stable, and the in-situ sloping slab was constructed, the shuttering being bolted between the frames. This slab was cast in sections, working up the slope with top shuttering.

The hillside slope was concreted in sections, working up the slope with a spade finish. The vertical division wall slabs were cast on the flat on the ground, one on top of another, with a paper layer between each slab. These slabs were erected by the 10 ton derrick, each slab weighing about 4 tons.

The slabs were held in position by scaffolding tubes. They stood on timber packers, and the sides of the slabs had holes at 12 in. centres to which was bolted the shuttering for the in-situ vertical columns and junction pieces. The reinforcement for the in-situ work was fixed after the slabs were in position. The timber packers supporting the slabs were boxed round and removed after the in-situ concrete had set, and the holes were subsequently filled up with concrete.

The erection of the precast wall slabs is shown in Figs. 20 and 21.

The work was programmed to complete the screen-supporting beamwork as quickly as possible, and for this purpose some of the side walls, beams, and junctions were not concreted until a later date. As soon as the first top beams were cast, erection of the screen house machinery commenced. Ferrocrete was used for all the precast construction, using a mix of roughly 1:1½:3. The total concrete in the structure was 500 cu. yds., and the reinforcement amounted to 40 tons.

Fig. 22 shows a general view of the completed structure.

Conclusion

The design and construction of the " Temporary Works," as they are generally called, is an important and interesting section of a contractor's work, and it has been the intention of this paper to give a very broad picture of the nature of these structural design and construction problems that are secondary to the main Contract Works on a heavy civil engineering project. It has not been the intention to analyse the detailed mathematical design or economics of the individual structures, but rather to give to the younger members a general background to a branch of structural engineering that is essential to the erection of large permanent structures, but seldom has the same publicity.

Acknowledgments

The Author wishes to thank the Chairman and Board of the Belfast Water Commissioners for their kind permission to present this paper; also the Consulting Engineers, Messrs. Binnie, Deacon & Gourley; and the Author's employers, Messrs. Charles Brand & Son, Ltd., for their assistance in the preparation of the paper.

Book Reviews

Elastic Waves in Layered Media, by W. S. Jardetzky and F. Press. (New York and London: McGraw-Hill, 1957). 9 in. × 6 in., 380 plus x pp. Price 75s.

The Authors of this book emanating from the U.S.A. are well known in geophysical literature and the character of the book is indicated broadly by their respective professions—Geology, Mathematics and Geophysics. The emphasis is, however, on mathematical theory but experimental and field aspects are also covered. The book will probably be of greatest service to the geophysicist investigating earthquake phenomena or carrying out seismic site investigations.

It is not easy to indicate precisely how the book can be of service to the engineer but, if a problem in structural or mechanical engineering involves elastic vibrations, a study of the text may prove well worth while. Most of the book is concerned with the propagation of elastic waves in layers of material but one chapter is devoted to wave propagation in plates and cylinders. The theoretical problems treated in this chapter include vibrations in plates and rods in a vacuum and in a liquid, vibrations in cylindrical tubes and the effects of empty and liquid-filled holes in elastic media. The theoretical considerations concerning vibrations in rods may, for example, be of interest in pile-driving studies.

The book attains a high standard and can be thoroughly recommended as a reference text for seismic studies and special vibration problems in structural and mechanical engineering. There are extensive lists of references at the end of each chapter to facilitate further reading.

F. D. C. H.

An American Architecture, by Frank Lloyd Wright. Edited by Edgar Kaufmann. (New York: The Horizon Press, 1955; London: The Architectural Press.) 12 in. x 9 in. 269 pp. 84s.

Frank Lloyd Wright is world-famous as a great architect, and in this book, which is really a collection not only of his designs but of extracts from his speeches, comments and letters, it becomes clear that he is not unaware of this himself. It is, however, stated in the editor's notes, " Genius is inseparable from opposition." Opposition has never daunted Mr. Wright, and his challenge to the accepted way of life is as fundamental as is his faith in the ideals and principles which formed and continue to occupy his work.

The book is most admirably produced, and gives one a real insight into the mind of one who has approached building design and construction from the viewpoint of certain human and technical principles which have no ties to any tradition.

To Mr. Wright freedom is the first essential in all things, and he has set out to provide buildings with an air of space freed of enclosing walls and with no sense of " box building."

His designs cannot fail to be of the greatest interest to members of the structural engineering profession, and demonstrate how structural design should be the servant of aesthetic design. Many of the works illustrated merge into their surroundings in a most delightful manner which he himself describes as " Organic Architecture—natural to the time and place for which it is designed, natural to the man for whom it is built —in fact, an architecture of basic principles."

A. H. L.

Differential Equations, by S. V. Fagg. (London: English Universities Press, 1956.) 4¾ in. x 7¼ in. 127 pp. 7s. 6d.

There is at present great interest in the subject of differential equations. The book, under review, is another one towards helping the student to a better understanding of this interesting subject. It is an elementary text-book; its appeal is at school level, but it is also suitable for young engineers reading for their professional examinations. It is brief: there are numerous practical examples, more for mechanical and electrical engineers than for structural engineers. In further editions, perhaps more examples could be given, which are helpful to the civil and structural engineering student. It could be enlarged to introduce a chapter on equations of an order higher than the second, and more particularly dealing with linear equations of the fourth order with constant coefficients.

The Design and Construction of the Prestressed Concrete Hangars at Helsinki Airport, Finland*

Discussion on the Paper by Dr. F. W. Gifford, A.M.I.Struct.E., and Mr. Matti Janhunen.

THE President introduced the Authors who then presented their paper. Mr. Janhunen also exhibited a film illustrating the progress of the work of construction and some of the details.

Mr. JANHUNEN, in thanking the President for his words of welcome, said he was very pleased to attend the meeting and to help in presenting the paper and dealing with the discussion.

The Helsinki hangars provided a very prominent example of the new building technique in Finland ; the structure was the biggest of its kind there.

He went on to discuss the work from the Finnish point of view and having regard to the conditions in Finland.

The precast elements for the secondary beams, he said, were made during the summer. The stressing, grouting and lifting of the big beams had to be carried out during winter, and that had given rise to many difficult problems which had to be overcome.

Discussion

The PRESIDENT proposed a hearty vote of thanks to the Authors, and opening the discussion said the Institution was to be congratulated on having their valuable paper for inclusion in the Journal. They had described one of the largest prestressed structures of recent years, involving many complicated design problems and, in addition, difficult climatic conditions and construction requirements. One of the facts which had struck him on reading the paper and studying the photographs, was that the wintry conditions experienced were such as might bring even a major mass concrete job to a standstill. Yet under those severe conditions relatively small and highly stressed concrete members were constructed and erected, in addition to the delicate and accurate requirements for prestressing and grouting. The engineers and contractors deserved great credit for their courage and determination to achieve the maximum progress, despite the hazards of a severe Finnish winter.

He congratulated both Authors on an outstanding contribution to structural engineering, and was sure there would be a very interesting and constructive discussion.

Mr. J. N. LOWE (Managing Director, the Pre-Stressed Concrete Co. Ltd.) made a contribution which was presented on his behalf by Mr. Rodin.

First, as one who knew the facts, Mr. Lowe congratulated the Authors on the presentation of those facts.

Discussing the shape of the secondary beams, he said he had been asked many times why they were not simple bow string girders. The reasons were two-fold. In the first place, when they had presented the scheme to the client it was thought that the triangular girders were more stable for lifting, and secondly that they lent themselves to using standard elements for the roof cladding. Just before the final scheme was drawn up his firm did in fact discuss with the client whether or not to revert to simple bow string girders, but by that time they had become so enamoured of the V-shaped hangers that for this and other reasons it was decided to adhere to them.

The questions of buckling of the main arches, of adjusting the ties of the main arches and the hangers from them, the methods of lifting secondary beams and, indeed, the general problem of the design were dealt with most carefully in the paper ; but there were two points he wished to bring out especially. There were many things that one learned from each design, and those two points could easily be under-emphasised.

The first was the fire that had occurred. There was, fortunately or unfortunately depending upon one's point of view, little experience of actual fires on prestressed concrete structures. As in all structural frames, what little experience there was had been connected with cases when the fire had been disastrous, resulting in a total loss of the structure. In the case of the Helsinki hangars the experience had not been calamitous and there might be considerable lessons to be learned from that particular instance : at all events there was more information now to hand for engineers to consider when dealing with problems of fire resistance. He emphasised that there was a beam which had been subjected to heat for a certain length of time which the eminent Finnish engineer, Mr. Varjo, had thought fit to accept as a permanent part of the structure. If there were any further interest in that fact, Mr. Lowe felt sure that his Company or the Authors could supply information.

The second point was that of casting, and particularly jointing, precast concrete units together for prestressing in very cold weather.

It would have been noticed that the contractor had prepared for cold weather by having the correct equipment almost as naturally as contractors in this country provided cement for concrete on site. The lesson to be learned from that was clear. It was cold in winter in this country too : many contractors forgot that concrete—and particularly jointing concrete—required longer to attain its full strength in cold weather than in normal weather, and the contractor, by forgetting that fact, could unwittingly upset his programme and destroy any advantages he had intended to gain from precasting.

Those two points Mr. Lowe wished to emphasise.

* *Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on 22nd November, 1956. Mr. J. Guthrie Brown, M.I.Struct.E., M.I.C.E. (President) in the Chair. Published in "The Structural Engineer," Vol. XXXIV, No. 11, pp. 415-427 (Nov., 1956).*

The Authors, he continued, had very kindly and courteously made acknowledgments to those who had permitted the paper to be written and who had assisted in preparing the paper in any way, however small. There was one name he would like to add to that list on behalf of his own Company because the person concerned, although he did not help in the actual presentation of the paper, had helped in the preparation of the design. He referred to the former colleague of himself and Dr. Gifford—Mr. A. J. Harris.

To summarise, Mr. Lowe said he would like to thank the Authors. He had explained why the triangular secondary beams were adhered to, he had laid emphasis on the extra facts about fire in prestressed concrete which had come to light and on providing for cold weather; and he added his Company's acknowledgments—as well as those of the Authors—to Mr. Harris.

Mr. A. J. Harris added a remark which gave the reason why some of the forms were adopted during the early days of the design. The shape of the main arch in particular, he said, might seem to some people to be slightly eccentric. The reason was that over the whole site a very good solid granite existed immediately below the ground level; it seemed economical to take advantage of it as far as possible to support some part of the arch thrust. A full parabola with the whole thrust carried down to the ground would occupy too much space, however, and a compromise was adopted whereby only a fraction of the thrust was carried by tensioned steel at eaves level and the balance was carried to the ground by an inclined leg.

There was one other aspect of the arch design on which Mr. Harris felt the Authors might enlarge, and that was the buckling problem.

The hangers were of small section and quite clearly they offered no stiffening against torsional buckling of an arch which, after all, was for 250 ft. span. That had caused considerable headaches at the time; there was, however, a factor favouring stability, and that was that as the arch tended to buckle transversely, so the inclination of load also changed in direction since the feet of the hangers were fixed in position, a state of affairs which was rather different from the classical buckling case where the vertical load remained vertical even after the arch was deflected. Acknowledgment was given in the paper to Professor Naylor, who had published a paper at a timely moment, and his assistance was very valuable on that particular problem.

The only other comment Mr. Harris made was that one of the pleasures of the job was to meet on his own soil Mr. Janhunen; not only was he a most charming man to work with but there had been much to learn from him. The skill and the conscience which he applied to the construction of this major structure were exemplary.

Mr. E. W. Gifford asked what structural assumptions were made in the analysis of the secondary beam girders. He supposed the top boom was assumed to be rigid right through and the hangers of the beam were assumed to be flexible; and he asked if that were the case and how far the tests had borne out the assumptions in practice. The deflection, compared with the calculated value, was unusually low and might indicate a slightly greater degree of redundancy than was assumed. Also, did the tests yield any further information on that type of member and on the validity of the design assumptions with regard to strain readings and points of cracking, for example?

Next he raised the question of creep losses in a member of that kind. Normally, he said, the near range of stress in a concrete beam, i.e., a beam rather than an open web structure of the kind under discussion, was much less than in the bottom boom of a truss. In the boom stress varied from 0 to 1400, whereas in the prestressed beam the mean variation was very little from one condition to another. So that it seemed that there might be some difference in the creep losses, which might result in important variations in length of the boom in a redundant member of that nature. He asked if long term tests were taken to determine that matter.

Mr. Gifford then referred to the ultimate load condition of a non-triangulated member where the stiffening was produced by the prestress of a tie. Under overload conditions the prestress could be removed from the tie by the direct tension, in which case it appeared that there would be a rapid breakdown of the stiffening member. It was probable that the conditions in which the stiffening member was required to function as a stiffener were not those obtaining when it was at its greatest load as a tie. He would be glad to have some information on that.

Coming to the joints of the rafter members where the continuity bars passed into the joints, he said he was not quite clear concerning the function of these. If it were to provide continuity in bending, was not the bond length rather short? If it were not to provide continuity, did it not make the joint somewhat more complicated to produce? Why did the Authors choose to use that type of joint as against a plain butt joint?

Those were perhaps small points, he concluded. But in view of the importance of that particular structure at Helsinki, the boldness of the conception of which was remarkable, he would be glad if the Authors could give further information on these points.

Lt.-Col. G. W. Kirkland, M.B.E. (Vice-President) said that the designers' conception of the hangars at Helsinki was most creditable, but there were two small enquiries he would like to make.

First, Mr. Janhunen whose skill and that of his collaborators was most noteworthy, had carried out the whole of the construction on timber scaffolding. In this country timber, being an imported material, was rarely used; but he was most interested to know if any particular problems were experienced in jointing the timber and to know what provision, if any, had been made for scaffold yield.

Secondly, he asked if the Authors would give information on experience in the tensioning of the cables.

He would be grateful for a resumé covering such items as frictions, extensions or other special experiences due to temperature variations.

Mr. J. Bak (Member), referring to details of canopy (Fig. 5) said that from the drawing on Fig. 5 it was not quite clear how the cables are fixed, and in fact what happens at the right hand edge of the drawing.

The beam at the right hand edge of the drawing should be apparently composed of 2 top and 2 bottom members and yet, there are a number of cables that are taken right through the depth, and he would appreciate some details of this point.

Again, he was not quite clear how the 28 x 2 cables were anchored in the middle column, and possibly a little explanatory sketch on this point would also help.

Mr. B. L. CLARK (Member) asked if the Authors could give figures concerning the deluge system which was used in the hangars, the volume used and the areas covered in the case of fire.

Next he asked what factors had dictated the span of the hangars. The span was not big enough to cover two big aircraft, and it was rather big for one single passenger plane at the present time.

Speaking of the doors, he asked for information on their construction and the mechanical method of operating them—whether it was push-button electrical control or otherwise. Again, what arrangement is adopted for opening the doors : were they on a number of tracks, and how much of the opening could really be used at one time ?

Mr. G. E. DUNCAN (Associate-Member) sought information concerning the concrete mixtures and the grout used at the junctions of the precast beams.

Mr. J. R. LOWE (Associate-Member) asked what precautions were taken to ensure that no free water remained in the grout after setting. He had in mind troubles experienced in cold countries, where cracking of the concrete had occurred due to the expansion on freezing of such free water.

Dr. GIFFORD dealt with some of the questions that had been put. First, however, he thanked the meeting for their kind reception of the paper.

Apologising to Mr. Harris for not having mentioned his name in the paper, he said it was only when it was in print that he had realised the name was not there. There were also some other members of the Prestressed Concrete Company whose names he would also like to mention, for they were associated with the work at various times. They were Mr. R. Strapp, Mr. R. Turner and Mr. A. Dishman.

The points that were raised by Mr. Ward could perhaps best be answered by Mr. Janhunen.

Coming to the remarks by Mr. Harris concerning buckling and other problems, he said that little information was available at the time the design was being considered ; but one experimenter did produce information at the time showing quite clearly the effect of the hanger tension changing direction as the arch buckled. It was rather interesting to note that a single arch, simply supported at the ends, did not just buckle in one phase, but must buckle in two phases, so that there was a point of contraflexure at the centre of the arch, and any attempt to reduce the buckling of the arch by fitting central restraining members was hopeless. On the other hand, an arch which was restrained at the end would buckle in a manner similar to a fixed ended strut. An arch, freely supported at two ends, could simply collapse by lack of equilibrium.

Dealing with Mr. Gifford's reference to the structural assumptions made for the secondary beam girders, the vertical triangular members were assumed to be rigid ; thus the deflection of the top and the bottom booms was assumed to be the same : the main stiffness was not in the top boom, but in the bottom boom, as that was prestressed.

Again, so far as design was concerned, they had taken into account the stiffness of the two top booms in addition to the bottom boom. They had made the assumption that the triangular system at the front was quite rigid, and likewise for that at the back.

As regards ultimate load conditions, the probability was that if the member were tested to destruction the factor of safety would be less than the factor of safety of a fully prestressed beam, when the ultimate load was divided by the design load. But in that particular case they were dealing with a member where the live load was comparatively small as compared with the dead load, and therefore the factors of safety required could be a lot less. If the dead load were 1 and the live load were 1, then an increase of 1 in the live load produced a much larger effect than when the dead load was 10 and the live load was 1. So that he did not think the ultimate load was of great concern.

The PRESIDENT, at this stage, asked if the Authors would be good enough to reply to the remainder of the discussion in writing. On behalf of the meeting he thanked them both again for their excellent paper, and conplimented Dr. Gifford on the manner in which he had dealt with the questions.

Written reply by Dr. Gifford

Mr. J. N. Lowe's remarks call for little comment ; regarding the fire, it was apparent that this was not of sufficient time length to do serious damage to the structure, the spalling being comparatively superficial. One of the main dangers was the possibility of damage to the steel at the rear, where it was exposed to the fire, but tests had shown that the steel was not affected.

Referring to Mr. Harris's remarks relating to the shape of the main arch it is of interest to note the effect that delay in the decision relating to the door construction materially affected the main structure. It was originally intended to have a door housing some twenty feet long on either side of the main openings and in consequence it was necessary to keep the raking leg back to avoid interfering with access to the hangar side. Subsequently the door housing was considerably reduced in size, and had this information been available at a suitable stage it is highly probable that the main arch would have extended tangentially to meet the rock, providing access below. This would have eliminated need for the horizontal tie force.

With regard to Mr. E. W. H. Gifford's remarks, the secondary girder was certainly appreciably more redundant than was assumed for the purpose of design—the vertical triangular members would resist bending although this was ignored. The tie acted as the stiffening member and resisted bending due to the cantilever and due to non-uniform live load, the prestress provided counteracting to a large extent the cantilever effect ; in addition to the cables provided to take the arch thrust, cables were required to provide pre-compression to allow for the worst possible bending due to non-uniform live load—this being assumed to be live load on one half span. Thus there would be considerable reserve of compression for uniform overload, and for uniform combined with non-uniform overload the stiffening member would only break down when pre-compression was completely eliminated from a part of the tie. This was considered to be highly unlikely.

With reference to the question of creep loss it is considered that the difference between an open web girder such as the secondary girders, and a normal type of beam is not material. As far as creep of concrete is concerned, provided concrete stresses are kept reasonable, the steel stress loss due to concrete creep is governed by the stress in the concrete adjacent to the steel, and also to the time the concrete is under stress. For this purpose it is reasonable to consider the centroid of the prestressed steel, and in consequence any stress variations across the tie become relatively unimportant.

Unfortunately as the testing of the fire damaged beam was carried out at the expense of the Insurance Company, they insisted that the time spent on it was kept to a minimum, and for this reason the measurements taken were restricted to deflections. No cracking was reported during the test.

Referring to the joints of the rafter members, these were not required to transfer bending ; the matter was considered bearing in mind that there is a difference between this joint and a butt joint which has cables passing through it, and that the fairly heavy girders had to be lifted a considerable height, and the decision was made to use a definite tie. Similar joints are currently used for similar open web structures, the cost being considered relatively small.

In reply to Mr. Bak, Fig. 5 shows in elevation the cantilever extension of the main tie of the secondary girders, shown to smaller scale in Fig. 3a and 3b. Thus the cables shown on elevation are those of the main tie which extend its full length and are anchored in the rear triangular anchor block, on either side of the supporting rear column. Referring to the cables of the main tie, at the central main column, a solid horizontal diaphragm was introduced at main tie level (see Fig. 2b) and the main tie cables were simply fanned out and lapped so that cables from the left hand span were anchored on the right hand face, and vice versa.

Written reply by Mr. Janhunen

To Lt.-Col. G. W. Kirkland

The main cables of the secondary girders, which were stressed from the both ends, had a sharp bend at the hangers. In order to reduce the friction the designer wanted to stress the cable in a manner to avoid sliding at this point, and calculated the elongations of the cable for the front end 43 mm. and for the back end 225 mm. Soon we however noticed that an equal increase of tension in both ends of the beam was necessary for a successful result.

The most usual reason for an unsuccessful stressing was wires crossing near the anchorage. On two occasions a spiral of a cable entered the female cone during stressing and prevented the male cone from firmly anchoring the wires. As the jacks were new there was a tendency for the wires to slip at the anchorage on to the Freyssinet jack, but with time the wires became more rusty and the grooves in the wedges worn, and this trouble vanished. During the stressing of 256 cables it happened on eight occasions that the wires of a cable either completely or partly slipped. This could happen after several hours and even some days after fastening the anchorage, the reasons being as mentioned above. It is possible that some male cones were not regular enough to keep the wires anchored. The " pull-in " at the anchorages was about 6 mm. in, each end.

Most of the long cables of the secondary girders had to be stressed in winter when the rear of the beam was in a heated room ; warm air passed into the cable channels where it caused condensation and the cables froze solid. It was impossible to release the cables before the ice melted during following mild weather.

At each end of the beam there was a tensioning group of two workers and a foreman. A fitter kept the equipment in order and marked the wires. The two tensioning groups had a telephone connection and an engineer supervised the work.

To Mr. B. L. Clark.

The mechanical method of operating the doors is push button electrical control. The doors roll on rails and $\frac{3}{4}$ of the door wall can be opened.

To Mr. G. E. Duncan.

The units were jointed with rapid hardening cement grout and aggregates (maximum size of grains were 8 mm.) in proportion 1:2.4. Watercement ratio was 0.42. The required 28 days breaking strength (the same as for the units = 450 kg/cm^2) was reached in 10 days. The tensile strength was checked by test units 15 x 10 x 80 cm. and after three days it was 47—61 kg/cm^2. All joints were tested with a hammer. Of 672 joints, only 9 were faulty and had to be recast.

To Mr. J. R. Lowe

The grout used was a cement water paste with a w/c ratio 0.45. No sand was used because it was difficult to grout adequately in places, using heat cement grout. In tests it was proved that the water amount which parted from the grout was about 6 per cent. During test drillings after the injection water was never found in the filled cable ducts. On the other hand there was in the partly empty ducts always some water which of course was taken away before the grouting.

The winter circumstances gave considerable trouble. It was necessary to build a temporary working shed around the 52 meter long secondary beams into which warm air was blown, to get the ice melted and the water test made. At injection the temperature was kept a bit above O'C, and after injection the beam was kept warm for at least 24 hours ; the shed was then removed to another beam.

The President, 1957-58 (*Continued from page 367*)

distinguished contribution to structural engineering science, will enhance its prestige.

He will make many new friends during his year of office who, like those who have had the good fortune to be associated with him in the various activities of the Institution, will be attracted by his modest and quiet manner and his readiness always to give from the wide store of his knowledge and experience. Members of the Institution in all grades will wish him well in the coming Session.

Structural Problems Connected with Palletisation*

Discussion on the Paper by Mr. Brian Scruby, M.I.Struct.E.

Discussion

THE CHAIRMAN, who opened the discussion, said that he had never before fully appreciated the modern development in the handling and stacking of materials. It was a rather frightening thought that, when going to the grocer and buying some sugar or 1 lb. packet of margarine, one was setting in train a vast mass of movement in palletisation, ending up in an enormous store of the commodity. Long before the war, the grocers seemed to manage without all these elaborate depots; but people were now probably becoming the slaves of mass production, mass storage, mass advertising and mass buying. In fact, judging from the film which had been shown, in about fifty years' time the housewife would drive down to the grocer's shop with a little fork lift truck instead of a shopping basket. That was the way things appeared to be going.

The Chairman expressed interest in the very large aluminium roof trusses at the Woking depot. Some years ago, when building an aluminium bridge in Scotland, he encountered many problems in connection with the use of aluminium. The truss at the Woking depot was over 200 ft. span, and even with a modest variation in temperature of about 40 deg. the movement would be something like $1\frac{1}{4}$ in. Had any damage been caused to the roof covering or the glazing as a result of the large movement that occurred in aluminium structures?

Riveting with aluminium was also a very difficult matter. If bolting was adopted, as appeared to be indicated by the slides, were they ordinary steel bolts or cadmium coated? The use of steel and aluminium in contact set up some very awkward corrosive problems.

Mr. SCRUBY, in reply, said that the movement in the truss had not shown any detrimental effect so far. As shown in the diagram, this was well catered for in the rocker bearing at one end. The paper described how each longitudinal wall and each gable end was divided into three parts with an expansion joint. At those joints, the purlins were slotted and bolted to allow movement.

So far as possible, any possibility of damage from movement due to temperature differentials had been accounted for; but the building was not heated and, therefore, the extent of the movement was to some degree minimised.

The rivets were in aluminium, but the bolts themselves were plated.

Mr. W. D. DEVEREUX, who referred to the differential expansion between the trusses and the sheeting and any trouble with the glazing bars, said that as all the sheeting and the glazing bars were aluminium there was no differential expansion between them and the structure.

Mr. E. J. FISHER pointed out that the Author had not dealt with the question of fire risk.

With regard to the limitation of superimposed load on any floor, he recalled that in 1925 a tea warehouse was designed to limit the exact number of cases which could be stacked on the floor to prevent overloading. That involved palletisation in a minor way and it was most effective. The height of the ceiling was limited to allow, say, six cases of tea to be stacked on top of each other—there was not sufficient room for a seventh—thus ensuring that the floor was never overloaded.

Mr. SCRUBY replied that in the matter of normal fire precautions such as hydrants, extinguishers, and so on, Unilever and particularly S.P.D. left nothing to chance. Concerning the cube of the building, however, discussions with the Surrey planning authorities and the Surrey fire brigade had lasted for six months. Obviously, the building exceeded the allowable cube, and for this reason the authorities pressed for the installation of sprinklers. For a number of reasons, however, these were not acceptable to the owners of the building, although not necessarily because of expense. Ultimately, it was agreed both by the Surrey County Council and by the Surrey fire authorities that the building was not an undue fire hazard and agreed to the installation of an alarm system to the local fire brigade.

Mr. W. J. HERBERT referred to the permissible bearing load on the floor of 7 cwt. per sq. ft., and suggested that in the case of an ordinary post pallet with a capacity of 2 tons per pallet, the use of an ordinary pressed steel foot would give some rather high bearing loads.

Mr. SCRUBY replied that at the depots in question there were no post pallets as such. If there had been, the floor would naturally have been designed accordingly.

In the case of the S.P.D. depots, the margarine, for example, was stacked on racks and the tubular rack was so designed as to spread the load with tubes at the base. He could give the assurance that the equivalent load was not above 7 cwt. per sq. ft.

A speaker said that he was intrigued by the very interesting and large span trusses. Was it really necessary, however, to use such a large span in the work described by the Author? Had he collaborated with the warehouse managers or the people responsible for palletisation to ascertain whether the obvious increase in cost of construction of such large trusses was justified? The speaker was not belittling the large trusses, because they were good and gave complete freedom for whatever one wanted to do inside the building.

* *Read before the Institution of Structural Engineers at* 11, *Upper Belgrave Street, London, S.W.*1, *on* 14*th March*, 1957. *Mr. J. Guthrie Brown, M.I.Struct.E., M.I.C.E. (President) in the Chair. Published in "The Structural Engineer," Vol. XXXV, No.* 3, *pp.* 93-106. *March*, 1957.

One of his own clients was extremely keen on fork lift trucks and lifted everything with them, using his trucks to carry four times the weight that they were intended to carry. This raised questions concerning the floor surface. What was used was not a granolithic, but a compound of 80 per cent granite chips in a bitumen asphalt called Lithocrete, with the result that the trucks, even fairly heavy fork lift trucks, had no effect whatever on the surface. They did not ripple it as one would expect asphalt to do.

Mr. SCRUBY replied that the large span truss was a very controversial subject. In this case not only had the deputy-chairman of S.P.D. been present and consulted, but there had been full consultation with the warehouse managers and the directors throughout the whole of the organisation.

He did not necessarily agree that the large span trusses resulted in so much more additional cost. In fact, in certain circumstances it could be proved that they were not as costly. Obviously, each case had to be treated on its merits. It should be borne in mind that if the depot or building was split up, additional foundations had to be provided. If they happened to be piled or otherwise expensive, they could easily outweigh the additional cost of the single span truss.

The head room inside the truss was not such an important consideration. In other words, it was possible to get as much depth in the trusses as was necessary to design the truss economically. No heat was required in the building and, therefore, there was no question of loss of heat because of the extra cube.

The various types of asphalt floor finishes are well known and a sample is about to be tried. In this case there were two objections to it. The first was the question of smell or odour, especially when dealing with such delicate materials as margarine. Secondly, there was the rather subtle point that no matter how hard the asphalt was, it still had a certain resilience and trucks were apt to bog down in it, so that the manpower required to overcome inertia was greater than when concrete and steel, and so on, was used.

Mr. L. E. WARD (Member) suggested that taking the cost of the Woking building at £4 per sq. ft. and assuming that an extra stanchion in the centre would occupy 1 ft. width, then to provide the same aggregate floor area would cost £4 per foot run of building. Evaluating this to the 200 ft. span of the building it worked out at approximately 5d. per sq. ft. He felt sure that with the extra column in the centre much more than 5d. per sq. ft. could be saved.

Secondly, he thought that concerning aluminium, the Author had contradicted himself. It was claimed that on 200 ft. span aircraft hangars with rigid knees there was a greater advantage in weight saving ; but this was not quite true. A 200 ft. span portal frame was equivalent to about 160 ft. span between points of contraflecture. Therefore, the 206 ft. span truss being simply supported was equivalent to about 260 span in portal frame construction.

Mr. SCRUBY replied that he was not altogether clear on the arithmetic that was produced to arrive at the answer of 5d. per sq. ft. In case Mr. Ward was suggesting that it might be better to split the span into two and have a centre row of columns, it should be emphasised that any sort of column anywhere in the building would have been a great disadvantage to the clients, whose instructions were that columns should be avoided, not at any cost, but at some cost.

On the question of aluminium perhaps the paper had not made the point clear. The truss was simply supported. If it had been fixed obviously it could have been designed in a more slender manner, and weight obviously saved. He was trying to prove the point that at 200 ft. and over, aluminium trusses, if fixed, might prove to be competitive with steel.

He had used 206 ft. span trusses, and if these had been fixed at both ends, it might have been comparable with steel.

In the case of Woking, the freely supported aluminium truss proved to be more expensive than structural steel.

Mr. L. E. WARD added that in the case of the 200 ft. span portal frame the effective span between points of contraflecture was about 160 ft. Therefore when designing a 200 ft. span portal, one was actually designing for eccentrically loaded columns with a 160 ft. span girder in the middle.

S.M.D. had twice obtained contracts for 200 ft. span hangars in competition with steel and reinforced concrete. If this could be done successfully with a portal frame, it should be done more easily on a clear span truss where greater dead weight was involved.

The CHAIRMAN said that Mr. Ward's comments would be dealt with in the written reply by the Author.

Mr. SNOW stressed that the undertaking had been a big one and quite obviously it could not have been carried out in the time without a considerable amount of team spirit. All successful endeavours nowadays were the result of good team spirit. In the present case, they had been extremely fortunate not only with the contractors named by Mr. Scruby, but also with the sub-contractors as well as the clients themselves. From the results, the work appeared to be quite simple, but the clients had devoted hours of thought in deciding the type of buildings and why they wanted them for particular reasons.

In spite of any argument concerning the cost of the structure, in the majority of cases clear spans were justified. With the absence of columns, it was possible to instal the machinery for any particular process from one side to the other, without serious inconvenience. The problem had been studied from the overall viewpoint of the clients' present and future requirements.

Another aspect to be emphasised was the speed with which some of the depots had been constructed. In speaking of this, however, it was necessary to remember the dates when the site was found and when all the various plans were passed by the town and country planning authorities.

In the case of Chelmsford in particular, from the date of getting the site to the date of handing over by W. C. French was a period of eleven months. The actual construction time was about six and a half months, the remainder of the time being taken up in chasing the local authorities. The smaller depot at Norwich was built by Carter's in five and a half months, a similar rate of progress having been achieved at Lincoln. Other depots, however, took longer to construct. The record is likely to be achieved at Belfast, where there is quite a big depot which includes some 400 piles on the site of a disused pit. The construction time, including all the piling, will probably be 5½ months. It is an example of team work by all the parties concerned, and a great credit to the contractors—McLaughlin & Harvey of Belfast and London.

Mr. S. JOHNSON said he understood that greatly through the tenacity and drive of Mr. Davis of S.P.D. the small manual stacker trucks now had nylon wheels instead of the steel wheels which in the past had caused a great deal of trouble. Could Mr. Scruby give any further information about them?

Mr. SCRUBY confirmed that nylon wheels had now been tried. The process had gone from steel, which obviously caused most of the trouble to floor joints and surfaces, to compressed fibre, and finally to nylon. It was a question of economics—how much the wheels cost to replace and how long they would last.

The nylon had now proved itself to be the right type of wheel for the pallet trucks and was being tried out with great success. Obviously, the question of whether a granolithic or any other floor would crack up in five or six years was yet to be proved. In something like eighteen months, the floors had not cracked.

A previous Speaker pointed out that he had not been criticising the large spans. When making these large spans and increased heights in a building suitable for palletisation, its usefulness was increased by reason of its considerably greater cubic content. From the client's point of view the storage area was greatly increased. Because of the additional cubic content which could be used, the cost per cubic foot of the building could go up.

He asked whether, for use in big spans, Mr. Scruby had considered the high tensile self-inhibiting steels which were now available in the thin gauges from the Steel Company of Wales. They were almost comparable to aluminium in stress and weight ratio and to copper bearing steel for the structure.

Mr. SCRUBY replied that he had not tried those steels.

Mr. KENT, who referred to the question of large spans and the additional cost of the steelwork, quoted the case of a single-storey building for use with palletisation. It was originally designed with a length of 540 ft. and four bays of 92 ft. 6 in. each. The type of truss described by Mr. Scruby was designed in the initial stages. The trusses were to be at 20 ft. centres with three rows of internal stanchions, 26 in each row, making 78 internal stanchions in all.

Subsequently, an investigation was undertaken to consider the effect of increasing the clear spans to 185 ft., thereby having only one line of internal stanchions. The truss designed for the 92 ft. 6 in. span was not the right type of truss to use for the 185 ft. span because the building was to be heated and the additional roof space would have put it out of court on the score of heating the space between the trusses.

The whole arrangement was accordingly turned round and the stanchions were placed in one row down the centre, with 60 ft. centres on lattice girders and umbrella cantilever roof trusses of 30 ft. each span in between. The result was a building 540 ft. by 370 ft. with only nine internal columns. The additional weight of steel per sq. ft. was 1 lb. and the cost of the steel about £100 per ton. The additional cost worked out at about 10.8d. per sq. ft. For a matter of 11d. per sq. ft. the additional mobility available to the client by having only nine stanchions in the floor space as against three rows of stanchions at 20 ft. centres was of tremendous value.

When moving loads by fork lift trucks, which required to be able to turn in all directions and stack their pallet loads on various outlying spaces, there was no question that from the ultimate point of view the greater the floor area that could be provided without increasing the cost out of all proportion, the more advantageous the project became.

Mr. CARTER said he had heard of all these wonderful methods being applied to other trades and wondered whether the Institution could give just a little thought to the possibility of applying them to the building trade, in which bricks and drainpipes, for example, were still unloaded by the old, painful methods. Cement, of course, was taken in bulk as far as possible. As a poor old man, he needed help.

The CHAIRMAN replied that the Institution would keep in mind that cry from the heart.

Mr. SCRUBY suggested that it was a matter for the Institution.

A member pointed out that Fletton bricks could now be obtained packaged in packets of 50.

Mr. CARTER added that bricks cost £1 per 1,000 to carry, which was not economic.

Mr. DEVEREUX spoke of the relative costs of the big aluminium structures. If the height in relation to the width was about 25 per cent, his experience was that portal frame construction was more economical in comparison with steel or concrete. If, however, the height was greater, an aluminium truss was to be recommended. This was borne out by the hangars at Heath Row, where, in competition with aluminium trusses and steel columns, he had been successful with aluminium portal frames.

His company had carried out both designs to ascertain which would be the more competitive. They had also found that the larger the clear span, the more competitive was aluminium with steel.

Mr. WARD declared that that did not answer his question, but amplified it.

Mr. DEVEREUX asked Mr. Scruby to give an indication of the relative cost of steel as against aluminium construction in general terms.

Mr. SCRUBY replied that to speak of costs before one's clients at an Institution meeting was always a difficult matter. He had mentioned in the paper that the actual additional cost on the project was equal to 5½ per cent. It must, however, be remembered that aluminium required little further maintenance, whereas structural steel obviously required it.

Mr. DEVEREUX said that that answer did not quite satisfy him and he was hoping to be given figures. He was interested in the extent to which aluminium had advantages over steel. Obviously, factors such as span and many other things were involved. Could Mr. Scruby say in general terms, for example, that it was twice as dear, or only half as dear?

Mr. SCRUBY replied that on the 206 ft. span the cost was approximately 25 per cent more than for the actual equivalent in structural steel.

The CHAIRMAN pointed out that in some cases there were definite advantages from the maintenance point of view. The elimination of maintenance in an aluminium structure was a big factor.

Mr. SCRUBY added that there was consequently freedom from interruption of the client's processes on the floor beneath due to maintenance work.

Mr. SNOW described an amusing incident which arose as a result of the paper. The British Standards Institution is at present preparing a document supported by many eminent architects and engineers, on modular co-ordination. He opposed this strongly because he regarded it as a complete waste of money. A very large committee is dealing with the subject and heaps of money is being spent on it. Upon enquiry, however, he discovered that all the money is being supplied by the European Productivity Agency and as no cost to this country is involved, he withdrew his objection.

The question of modular co-ordination is nothing new. It has been going on for years. The paper in fact described a series of buildings together with all equipment and installations designed basically around a half-pound packet of margarine or soap. Similarly, other places were designed on the modular principle according to the sizes of bricks or shovels, or the width of trenches, etc.

The CHAIRMAN, in closing the meeting, said it had been clearly demonstrated that the great advantage of such large open floor areas in the warehouses was that they could be used for many other purposes—if necessary, in the national interest in the event of an emergency.

The fact that there had been a very good and interesting discussion was always an indication of a first-class paper. Everyone would agree that it was indeed an exceptional paper. It introduced a modern subject with which most members would now be much more familiar as a result of the paper.

Written Discussion

Referring to the relative cost of aluminium and steel, Mr. L. R. Creasy (Ministry of Works) recalled that in the early part of 1954 tenders were invited on a functional specification for a 200 ft. span hangar building from manufacturers of different types of construction including steel framework, aluminium, tubular framework and prestressed concrete.

The cost of the lowest aluminium frame was equal to the cost of the highest steel frame, but the lowest steel frame and the tube frame were each 30 per cent lower than the aluminium.

In the case of steel frame the cost of painting amounted to 20 per cent of the aluminium frame, and where allowance was made for this over a number of years it was not difficult to show that aluminium is an economical form of construction.

This adding-in of the maintenance into the first capital cost was a somewhat unsatisfactory device, and he thought the general conclusion would be that aluminium is not an economical form, at least for 200 ft. span hangars.

In the steel frame and aluminium forms there were 420 tons and 98 tons of metal respectively.

Mr. C. H. Bunclark (Member) referring to the discussion on the clear span of 208 ft. at Woking thought there was one point in favour of the clear span which was not mentioned—that was in reference to the extra piles which would be required if the clear span was sub-divided.

From the pile layout drawing he felt that possibly in the majority of cases single piles would have done from the point of view of carrying the load, and that maybe where pairs of piles occur, these were introduced from the point of view of lateral stability. Consequently, if the single span had been sub-divided into, say, three spans of 69 ft. approximately, the same number of piles would have been required for the circumference of the building, but extra piles to carry the interior columns. These extra piles, together with the necessary pile caps, would have run into several hundred pounds and consequently would have gone towards off-setting any saving in the steelwork by using shorter spans. Quite apart from this, in any case, the large clear span had such obvious advantages in such buildings that he was surprised so many questions were asked concerning the financial aspects.

Mr. SCRUBY replied that this point is very well understood and agreed that where depots have been piled it could be proved that a single span is cheaper than having a centre row of columns, and in the particular case of Messrs. S.P.D. Ltd., the open floor is an extremely important factor.

Mr. Scruby's written reply to Mr. L. E. Ward's comments on the paper.

Having had an opportunity to read Mr. L. E. Ward's comments, Mr. Scruby understood and agreed with him on the comparison between free spans and portal frames when considering the merits of aluminium versus steel. He was sure that Mr. Ward would also agree with him that the issue is also dependent on a number of other factors, particularly height of the columns and the dead to live load ratios.

His paper intended to point out that whilst it is generally believed that aluminium for spans over 200 ft., particularly in portal frames, is competitive with structural steel, his firm had proved that in the case of S.P.D.'s Woking Depot, a freely supported truss of 206 ft. span in aluminium was approximately 25 per cent more expensive. It must, however, in fairness be emphasized that the saving in maintenance costs and inconvenience was more than a compensating factor.

Post-War Developments in German Steel Bridges and Structures*

Discussion on the Paper by Mr. G. B. Godfrey, A.M.I.Struct.E., A.M.I.C.E.

Corrections

The Author, in presenting his paper, stated that the upper half of the table given on p. 67 was actually Table 6, which should have been inserted on p. 61 where reference to it was made. The lower part of the table had not been given any number as it formed part of Clause 2 of the German Specification for high tensile bolts.

He stated that there were also a few errors in the spelling of the German titles in the bibliography, which he hoped would cause no inconvenience.

Discussion

Mr. J. D. Vaughan (Member) showed some illustrations of the Westphalia Hall at Dortmund, in which five firms had been engaged, and gave some facts concerning the time taken over the job. The enquiry was issued in March, 1950. All the designs were made and completed before the order was placed. The order was placed on 27th December, 1950, or approximately nine months after the enquiry was issued. Steel erection began on 20th March, 1951, and was completed on 22nd June, 1951, or six months after the order was placed. This record looked very good—at least, the British idea of dates appeared very bad by comparison—but it should be remembered that the Germans never worked singly. Five firms had been concerned in the job.

Mr. Vaughan then spoke of the stiffened steel deckings. He had been to the M.A.N. works and seen the new bridge then being built over the Main, just west of Frankfurt. It had a continuous girder and a total span of 253 metres. There were five bays, measuring 39, 50, 75, 50 and 39 metres. On his visit in May, he was told that the job was begun on 1st April and would be finished on 15th June, and he knew sufficient about the German methods to be sure that they would finish on that precise date.

It was a single autobahn with a 7.5 metres span, the main girders being 2.99 metres overall. The steel decking was put in in sections and was stiffened by steel flats, and the cross girders were 3.65 metres apart. An interesting point was that horizontal stiffeners were used on the main girder and they were made of channels. They were not continuously welded to the webs. The welding was intermittent—4 in. welding, 12 in. spacing. To ensure that the welding did not exceed these figures, the web of the channel member connecting to the main girder web was cut away.

Mr. Vaughan then showed slides of a bridge which had been commenced in Cologne. As usual, a prize was given for the best design, but the bridge chosen was not the first prize-winner. It was a suspension bridge, with only one tower, having tramlines, carriageway, cycle tracks and footpaths. It had a height of 64 metres and a span of 302 metres.

Mr. P. S. A. Berridge, M.B.E., speaking of the remarkable advances in German design, said that so far as medium span railway bridges were concerned British design was not lagging behind Continental practice. He did, however, draw attention to the permissible stresses being specified in the draft revision of B.S. 153. If these were to be lower than the German equivalent for a steel having a lower ultimate breaking stress than that used in this country, it followed that the British bridge was bound to compare unfavouraby with Continental design. The design shown in Fig. 6 of the paper where the stiffeners were welded to the tension flanges was presumably based on the permissible tensile stress for basic loading of 10.16 tsi for a mild steel having an ultimate of 23.5 tsi. Compared with the British equivalent where the permissible stress would be 9.0 tsi and the steel would have a minimum breaking strength of 28 tsi, the German bridge had a lower factor of safety and would therefore be cheaper.

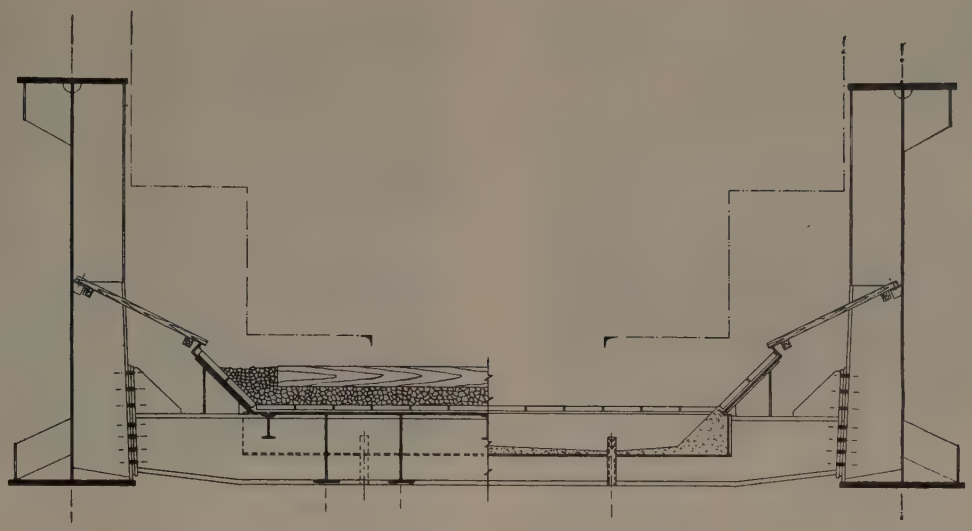
In the draft revision of B.S. 153, Part 3, the permissible stress was reduced by a factor depending on the ratio of the minimum to the maximum loading and on the numbers of cycles of loading to be expected in the life of the bridge. The lighter the bridge, the lower would become the permissible stress! In the half-through type railway bridge the welding of the intermediate stiffeners to the bottom flanges could, when the design was in accordance with these restrictive permissibles, result in increases in the sectional areas of the tension flanges of girders for spans of, say, 50 and 80 ft. by no less than 49 and 32 per cent respectively.

Some reduction in the permissible stresses in welded design might be desirable to guard against the possibility of fatigue failure; but such a reduction should not be made unless there was concrete evidence to show it was necessary. It appeared that the Germans paid no attention to such a requirement; and if British designers were to be able to compete in the world's markets it was imperative they should not be shackled by permissibles that were unnecessarily conservative.

Mr. Berridge thought the relative increase in drawing office staff compared with the numbers of men on the workshop floor was probably due to a reduction in the latter following a change-over from riveted construction to welding. That change-over had reduced the work in both the drawing office, where much less detailing was necessary, and the workshop where the elimination of centre-punching, punching, drilling and riveting made an even greater difference.

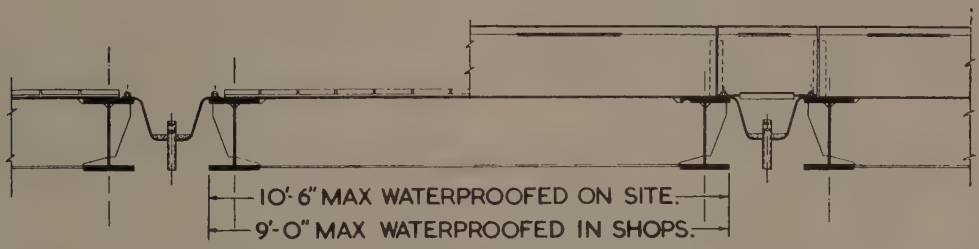
Mr. Berridge doubted whether the 4 cm. of asphalt under the ballast was adequate protection for the steel plating in the design shown in Fig. 6. If corrosion did set in there, and it was going to be very difficult to stop if cracks occurred in the asphalt, the strength of the whole span would be impaired.

* *Paper read before the Institution of Structural Engineers, at* 11, *Upper Belgrave Street, London, S.W.*1, *on the* 14*th February*, 1957, *the President* (*Mr. J. Guthrie Brown, M.I.Struct.E., M.I.C.E.*) *in the Chair. Published in* The Structural Engineer, *Vol. XXXV, No.* 2, *pp.* 53-68.



HALF CROSS SECTION THROUGH UNIT.

HALF CROSS SECTION THROUGH TROUGH GUTTER.



PART LONGITUDINAL SECTION.

Fig. 1.

Fig. 2.—Half through type Railway Bridge under fabrication for the Western Region, showing the prefabricated Steel Deck Units prior to Waterproofing

Fig. 3.—Another View of the prefabricated Deck Units for the Western Region Bridge, showing the Single Troughs between each pair of Units. The Troughs reduce the effects of floor interaction and facilitate drainage

The placing of the gussets in the same plane as the chord plates in the welded truss shown in Fig. 16 was not entirely novel. W. T. Everall had adopted that feature some time ago in his welded military bridge in this country.

Mr. Berridge then showed three slides, from which Figs. 1, 2 and 3 have been produced, to prove that design in Britain was as much to the fore as anything that had been shown from Germany. In the latest half-through type railway bridge with prefabricated steel deck units, painted and waterproofed before delivery and ready to drop in on shear plates on the sloping flanges of stiffeners on the welded main girders, the erection work was confined to the relatively simple expedient of tightening small numbers of high strength bolts carrying the end fixing-moment stresses due to fully rigid joints between the floor members and the main girders. The ballast was contained within units fully protected, while steel troughs between adjacent units provided excellent drainage and, at the same time, served to prevent serious floor interaction between the main girders and the decking. All exposed surfaces of the girderwork were easily accessible for maintenance.

Mr. O. A. KERENSKY (Member) suggested that he might have prompted the writing of the paper, as for many years he had been pleading with the Author and with various influential members of the steel industry to make available translations of German publications and specifications. After the war the Germans had an exceptional opportunity to develop bridge design and methods of construction, and the Author has now helped to fill the gap in our knowledge of this development. He hoped that the steel industry would make arrangements to provide regular information about what was going on in Germany and elsewhere.

Referring to Table 1 in the paper, it seemed that the Germans now had a greater variety of steels than Britain, and yet the British high tensile steels were some of the first to be commercially developed in the early 1930s. It must, however, be noted that the German steels had a much smaller plastic range. A comparison of Britain's structural weldable steel 968 with one of the best German steel was as follows :

	B.S.968	German H.S.B.55
Carbon	0.23%	0.20%
Mn.	1.8%	0.9%
U.T.S.	33-43 T/sq. in.	35-43 T/sq. in.
Yield 16mm.	21 T/sq. in.	29 T/sq. in.
Yield 30mm.	19 T/sq. in.	28 T/sq. in.
Elongation	18%	20%

Perhaps the Author could say something about the prices of the German high tensile steels. There were some very good special British steels on the market but their cost was prohibitive for normal structural purposes.

One must note that the Germans had already produced specifications or rules for composite construction of steel and concrete and for the use of high tension bolts. Generally, their specifications were complex and detailed and could be used only by competent, highly-trained engineers.

He disagreed with Mr. Berridge that the number of designers had increased twofold relative to fabricators because of the introduction of welding. Most of the bridges which had been illustrated were riveted, and he did not think that the shop efficiency could have improved by 100 per cent or that the efficiency of the designers had been reduced by 100 per cent. Any increase in the number of designers relative to fabricators was because the designs had become so complex that additional manpower was needed to produce them. The paper makes it clear that economical structures cannot be conceived and designed without much labour. Mr. Kerensky had calculated that, at today's prices, it would pay to employ a competent designer for a month to save a ton of steel—and he did not know of any designer who could not do that in a month ! He also assured Mr. Berridge that the proposed rules in B.S.153 dealing with fatigue are based on German and American work, and that it was certain that neither in Germany nor in America were railway bridges on busy lines being designed at maximum basic stresses.

Looking at the bridges shown in the paper and in the film, Mr. Kerensky said he was full of admiration for their appearance, although he could not see the economic reason for most of them. He did not understand the use of very large plate girders, which in theory were not economic ; and although the Germans had achieved big savings in weight as against pre-war figures, he did not think that their weights were, in fact, the minimum.

On British prices steel decking was not economic, except for the largest of spans. The composite steel and concrete deck construction would win on cost, and probably also in quality, because the steel battle deck required more maintenance.

In his opinion, in all bridges illustrated by the slides, the spans were not big enough to justify the steel deck, certainly not at British prices.

On the Rhine, the Germans were using enormous cranes capable of handling pieces weighing hundreds of tons. These cranes were specially built for dealing with bridges demolished during the war. For normal work, and particularly for work abroad, however, such large cranes were not available and heavy chunks of steel were therefore not suitable for transport or erection. He predicted that the large plate girder bridges would not become fashionable with the British designer.

The cellular box construction adopted in many German bridges was ideal only in combination with plate girders as the local bending of chords of trusses affected the economy of the design. The paradox, therefore, was this : for short spans, plate girders were the correct economic and aesthetic solution, but the steel floors did not pay and therefore composite deck rather than the cellular box construction usually proved to be economic.

For long-span bridges, trusses were lighter and much easier to erect than plate girders, and steel decks should be economic ; but it was difficult to unite the two into a monolithic structure—hence, usually it was found expedient to provide cross girders and stringers to carry the deck independently of the main girders.

Commenting on the slides, Mr. Kerensky observed that the Germans were using one-sided stiffeners, about which there had been some controversy among British designers, but which were now permitted by the proposed new British Standard Specifications. He was somewhat surprised at the amount of welding per ton of steel : about 2½ per cent appeared to be a very high figure indeed.

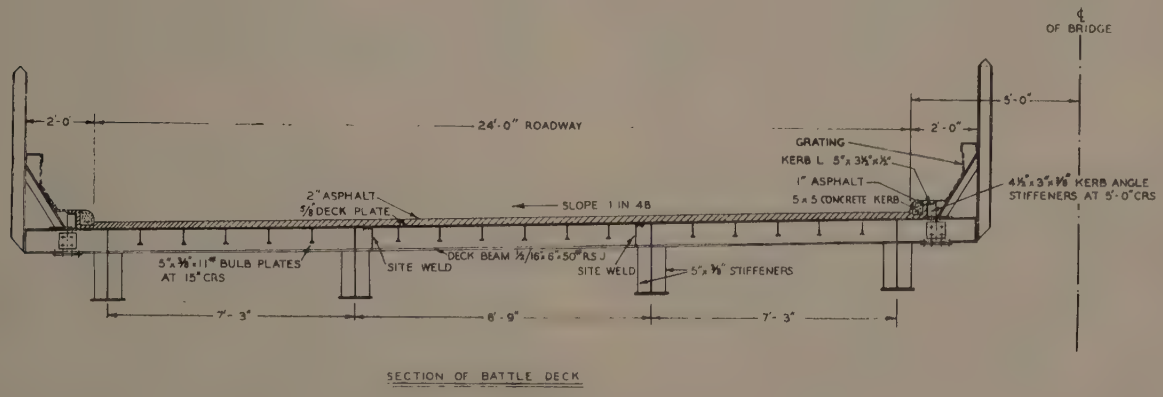
Having seen the pictures of the new German bridges, one was perhaps inclined to feel envious ; but on further study there was some satisfaction in finding that although we in Britain had not had the same opportunity for building bridges, our techniques of design and fabrication had not, in fact, lagged behind. Thus the composite construction had been developed shortly before the war, and there were many bridges designed by British engineers all over the world in which it was

Fig. 4.

Fig. 5

used. He showed two slides of bridges built in 1950 using composite construction. (Figs. 4 and 5.)

The reinforced steel plate deck was developed first in America and later in Great Britain and Germany. The Americans mostly used a steel plate welded to rolled beams placed longitudinally. Tables and data for this type of construction, to carry American loading, were available as early as 1938. In Britain, an all-welded deck for long-span bridges was developed in 1949 (Fig. 6). A panel of such a deck was fully tested as regards load-carrying capacity at the Building Research Station in 1950 (Figs. 7 and 8). Another test panel had been laid in Colnbrook By-Pass, and after being subjected to very heavy traffic for six years it was still in perfect condition.



SECTION OF BATTLE DECK

Fig. 6.

Fig. 7.—Model Deck under Load

Fig. 8.—View of Model Deck from Below

In that test panel, the surface of the steel plate was sand-blasted and metallised before asphalting—i.e., it received somewhat similar treatment to that adopted by the Germans for the Werratal Autobahn Bridge in 1952. The 1½ in. thick mastic asphalt surfacing was developed by the Road Research Laboratory in co-operation with Messrs. Limmer and Trinidad, and fully complied with the requirements laid down by the Engineers. It was interesting to note that it did not appear to be necessary to weld ribs to the steel plate to prevent the asphalt from sliding.

British Railways had adopted the American type of high tension bolt some years ago. Also several large bridges designed by British Engineers had been, or were being, built abroad using tensioned bolts in some if not all of the connections.

Referring to the German specification for the use of these bolts, the following observations could be made. The surfaces in contact were flame-cleaned. This should increase the friction between them and hence the value of the bolt by about 50 per cent. Equating a grip bolt to a rivet was in that case somewhat conservative. He believed that the latest German specification gave rational rules for calculating the value of the bolt in terms of induced tension, which was the only correct approach. Perhaps the Author could confirm this.

A clearance of 1 mm. in the hole was very small and, if used, any slip would be of little consequence as the bolts would almost immediately come into bearing contact with the parts connected, and act as pegs. As their shear value was very high, most likely the ultimate strength of the joint would be determined by bearing on the parent metal, as in some riveted joints. The recommended bolts were not of very good quality and the tension induced in them was fairly low. Better bolts were available in Britain.

He believed that the correct development of this method of connection was to aim at the least possible number of bolts of a given size, i.e. at the best quality bolt tensioned to its limit. This should prove economic and also result in the greater efficiency of the joint.

The use of polyester compound for sticking the parts together was worth special notice as it might have a great future in certain forms of construction.

Mr. D. Jubb (Graduate) spoke of the type of structure known in Germany as the Klönne/Silberkuhl roof, a latticed arch made of steel supporting a concrete shell, which is now being used in Britain. He had been to Germany to study the arch and had brought the design to British Standards. In the weight of steelwork in the arch, there was an increase of approximately 6 per cent when based on English stresses. (Mr. Jubb then showed a number of illustrations of this type of structure.)

The Chairman, in closing the meeting, regretted that time did not allow other members to contribute to the discussion. He thought it would be a good idea to have a paper by a British author on steel design and construction in Great Britain in order to show that this country had also some very advanced structures comparable with what had been shown by the Author.

Mr. J. McHardy Young (Member) in a written contribution said an interesting feature of the paper was the emphasis on plate girder construction; this raised the question of the limiting economic spans for this type of construction. Reference was made to the bridge over the Sava at Belgrade where the span is 855 ft. centres of piers. This would appear to exceed the limit of useful span for plate girder work and Mr. Godfrey's comments on this, with reference to the ratio of deflections to span, would be appreciated. It may be noted that the maximum span of the Neath River bridge is 300 ft. and that of one of the spans on the New Jersey Turnpike bridges 375 ft. Another interesting feature was the extensive use of horizontal web stiffeners which are rather novel in British practice although they were used in the Neath River bridge[54] and in the bridges over the Tigris at Baghdad.[55]

A number of the bridges have concrete decks designed on the principle of composite construction with corresponding economy in the steelwork in the superstructure. One advantage of using this type of deck (where the slab has a reasonable degree of flexural rigidity) is that the slab is effective in transferring point or live loads over any one supporting member to the adjacent members.

Mr. Godfrey also referred to the use of high strength bolts instead of site rivets, and in introducing his paper he illustrated a torque control wrench. This method of connection has been in use here for some time, but cases have occurred where the threads of the nuts have been stripped even though the correct torque multiplying spanner was used—and Mr. Godfrey's comments on the application of such bolts would be very useful.

Mr. Godfrey also illustrated the use of prestressed steelwork with either internal or external cables, and it may be that this method, particularly when used in tubular construction, will prove economical provided that the cables are protected against corrosion and loss of prestress. Perhaps he would be good enough to discuss the effect of shock loading on tensioned cables, as this had been known to produce failure in at least one case.

Mr. B. E. S. Ranger (Associate-Member of Council) was particularly interested to read that part of the paper dealing with the use of High-strength Bolts; but following the most interesting descriptions of projects carried out in a highly technical manner, it came as an anti-climax to read Clause No. 6 in the Karlsruhe recommendations for the use of such bolts, quoted by the Author on pp. 66 and 67.

This recommendation states that until further tests have been carried out, bolted joints are to be rated as riveted joints, that is, using one H.S. bolt for one rivet. This assumes that one H.S. bolt develops the same frictional resistance as one rivet in shear.

For American high-tensile bolts of ultimate strength about 50 tons per sq. in. and assuming a working load coefficient of friction of 0.25, this is approximately true. Even so, it had been reported[56] that the United States Research Council on Riveted and Bolted Structural Joints had stated that the "one bolt for one rivet" specification was conservative; also that the "Industrial Fasteners Institute of America" had stated that "two bolts will safely carry the shear load of three rivets." The German bolts referred to in the Karlsruhe recommendations are in steel of U.T.S. 63 tons per sq. in. and there are in this country high-strength bolts suitable for structural work of several grades of steel up to 65/75 tons per sq. in. These bolts will obviously provide higher clamping forces and therefore higher values of frictional resistance.

A great deal of research work has been done in America and elsewhere, including this country, into the value of friction connections in structural work. In view of the fact that there was likely to be a develop-

ment of the use of H.S. bolts in this country, we should approach it in the right direction and determine reasonable friction coefficients related to surface conditions rather than adopt at the outset this arbitrary rule-of-thumb of one bolt for one rivet. If necessary, coefficients on the low side could be specified at first; these could then be raised as the result of experimental work to date are examined and further work progresses.

He would like to ask what the Author's views on this subject were. It would also be of interest to know whether the bridge connections illustrated in Fig. 16 in the paper had, in fact, been designed on friction coefficients (and if so what they were) or on the Karlsruhe recommendation.

Mr. J. S. SHIPWAY (Graduate) referred to the term "bridle-chord bridge," which must be new to many engineers. Perhaps the Author would define it fully, making clear the difference between that type and a normal suspension bridge, and also if possible quote some other examples of the type.

Would he also give his views on whether it is feasible to prestress a bridge of the Köln-Mulheim type by first making the stiffening girders continuous between the anchorages and then shortening certain of the suspender rods by small amounts? Prestressing in this fashion would put more stress into the cables, but from Table 5 the steel in the cables is only 16 per cent of the total steel in the bridge, and it may be that although prestressing would require heavier cables an overall economy would be effected.

Lastly, it was of interest to note that the diagonal plates at the junction of the webs and flanges of plate girders used by Professor Dörnen were a construction employed by I. K. Brunel in his bridge work over 100 years ago.

The AUTHOR, in reply, thanked all those members who had taken part in the discussion.

In preparing his paper he had merely tried to chronicle the major events which had taken place in the German structural steel industry since the war. He had not attempted to compare German and British methods or productivity and he would not be drawn into any such discussion.

Space and time had been insufficient to enable Mr. Godfrey to give more than a brief description of some of the major bridges and structures, although he had provided an extensive bibliography to enable readers to pursue any facet of construction which particularly interested them. Consequently, he was grateful to Mr. Vaughan for his detailed descriptions of the Westphalia Hall, Dortmund, and of the bridge near Frankfurt.

Steel Decks.

To the Author the main point of interest about the Frankfurt bridge was that a steel deck had been used, although the maximum span was only 247 ft. and the minimum as little as 128 ft. Mr. Godfrey had been informed in Germany that steel decks were uneconomical for spans below 200 ft. For smaller spans it was suggested that composite construction should be used.

It may be of interest to record that the two separate superstructures for the Werratal bridge were almost identical in price, although the average span was 273 ft.

It was remarkable that the steel deck shown by Mr. Kerensky in Figs. 6, 7 and 8, which was developed here quite independently of the research undertaken in Germany, should be so similar in almost every respect to the decks being used on the Continent.

Steelwork Design.

Both Mr. Berridge and Mr. Kerensky had discussed the permissible stresses specified in the draft revision of B.S. 153 vis-à-vis the appropriate German specifications. It was certainly true that the permissible tensile stress for basic loading was 10.16 tons per sq. in. in Germany, but the Author believes that this is the only respect in which in practice the German specifications would be more favourable than the British. For example, at the supports of a continuous bridge, where the British specification allows both high bending and high shear stresses, the German specification would limit one of these stresses because the permissible equivalent stress, derived by using the Huber-von Mieses-Hencky formula, is as low as 11.43 tons per sq. in.

Mr. Berridge had expressed some fears that the Germans did not consider the effect of fatigue in the design of railway bridges, but the Author could assure him, as Mr. Kerensky had already done, that this was not so. The question of fatigue was dealt with in some detail, for example, in Clause 25 of BE804, issued by the German Federal Railways.

Although site connections are normally made with bolts or rivets, recently welding has been used very extensively in the fabricating shops in Germany, as a result of which it may well be, as Mr. Berridge had suggested, that there has been a reduction in the number of men employed on fabrication. Nevertheless, the Author's information is that a far greater number of skilled designers are required to produce the extremely complex designs for bridges.

Price of Steel.

Mr. Kerensky sought information on the price of German high tensile steels. In April 1957, the German Steel Information Centre kindly provided the Author with the figures for open-hearth plates given in the table. The price of sections was somewhat less.

TABLE OF STEEL PRICES

	Basic Price	*Extras*	*Total*	*Comparison*
St 37	£43.40	£0.32 (not weldable)	£43.72	100.0
		£0.50 (weldable)	£43.90	100.5
St. 52, HSB 50 or FB 50	£43.40	£11	£54.40	124.3

Silberkuhl Trusses.

The Author had actually known of the existence of Silberkuhl trusses, as mentioned by Mr. Jubb, but he was very pleased to have the opportunity of seeing some examples of this form of construction and interested to learn that the method is now being used in Great Britain.

Plate Girders.

Both Mr. Kerensky and Mr. Young had commented upon the extreme length of some of the plate girders. Both questioned the economy of such spans.

In considering such girders quite a large number of factors must be borne in mind. None of them are pure plate girders. They are all deep web-plates forming part of a superstructure comprising a steel deck or cellular or composite construction. All are in continuous construction. Consequently, the superstructures are stiff and the deflections not unduly large. The deflections are large enough, however, to warrant the use of trusses for railway bridges.

It should be remembered, also, that such structures are designed in competition with other forms of construction. Before the new Rhine bridge at Speyer was built, a competition was held in which 112 designs were submitted, incorporating a dozen types of construction. The spans are 171 + 536 + 351 ft., and although aesthetics played a part in the choice, the bridge selected and built was of box-section, not unlike that in Fig. 5 in the paper.

Of course, economy can be assessed in a number of ways. Normally, it is based on the overall cost of a structure during its estimated lifetime. When there is a shortage of steel, it may well be based on the amount of steel which will be used. In this respect, plate girders are not disadvantageous.

It is interesting to consider the tonnages employed in the Sava bridges in Belgrade, mentioned particularly by Mr. Young. The old suspension bridge, built in 1930-4, contained 6,800 tons of steel. The spans were 245 + 855 + 245 ft., and when the superstructure was destroyed during the war, the piers and abutments remained more or less intact. The Yugoslav Government therefore invited designs and tenders based on the old spans. Thirty-two designs were submitted from five countries, the successful design being that put forward by M.A.N. The new superstructure, details of which have now been published,[57] contains 2,400 tons of mild steel in two main plate girders, which were fabricated by two local firms, and 1,400 tons of high tensile steel deck, fabricated in Germany by M.A.N., the total amount being 44 per cent less than that in the old bridge.

This bridge is somewhat exceptional but, nevertheless, a reasonable maximum span for deep web-plate construction must be about 750 ft. Cable-braced bridges, discussed below, have been erected between 600 and 1,000 ft. in span, while two suspension bridges have been erected over 1,000 ft. in span.

Suspension and Bridle-Chord Bridges.

Mr. Shipway asked for certain information on suspension and bridle-chord bridges.

Pure suspension bridges have been erected at Köln-Mulheim and Köln-Rodenkirchen, the latter with spans of 309 + 1240 + 309 ft. Such bridges have cables extending from one end of the bridge to the other, the ends of the cables being anchored in the abutments.

For spans below 1,000 ft. Professor Dischinger developed cable-braced bridges, which in certain instances closely resemble pure suspension bridges in appearance. One of the most simple examples is the bridge completed by Demag in 1955 at Strömsund in Sweden, details of which are shown in Fig. 9, by courtesy of *Demag News*. The high tensile cables are fastened to the top of the pylons from which they extend to anchorages in the main stiffening girders.

The diagonal cables were pretensioned (as shown in sketches 7 and 10) in accordance with Professor Dischinger's patented design, for which the following advantages are claimed :

(1) Moments due to dead load can be considerably reduced or eliminated ;
(2) Moments due to live load are reduced because of the elastic support at the anchor points of the cables ;
(3) The pretensioned cables give much greater stability to the entire system ;
(4) From the economic point of view, the system is particularly adapted for cantilever erection since the erection system constitutes the final system.

The bridge now being erected in Cologne, which was mentioned by Mr. Vaughan and which is called the " Kontrapunkt " bridge, will have one pylon with three sets of diagonal cables, the two main spans being 990 + 495 ft. This is, of course, a very unusual structure.

The North Bridge in Düsseldorf, now nearing completion, having main spans of 354 + 853 + 354 ft., is in ' harp ' construction. This picturesque term is used because the three sets of cable bracing used with each pylon are parallel, the bracing being fastened to the pylon at the third-points above the deck and at the top. Preliminary details have been published.[58]

In the case of the Duisberg-Homberg bridge, the method of erection was almost identical with that of the Strömsund bridge. The temporary straight diagonal bracings which were used for erection were removed after the permanent curved bracings, called bridle-chords, and hangers had been installed and pretensioned.

The cables and hangers of the Köln-Rodenkirchen suspension bridge were also pretensioned and descriptions of this work have been published.[59]

Prestressed Steelwork.

Prestressed steelwork is such a new method of construction that there is little practical experience on which to base a reply to Mr. Young. Indeed, the most comprehensive scheme employing external cables, the flood spans for the North Bridge in Düsseldorf,[58] has only just been completed. In this, as in the Montabaur bridge mentioned briefly in the paper,[33] the cables closely resemble those used in suspension bridges. There is no reason to suppose that such cables will corrode or lose their prestress or suffer unduly from shock loading. Perhaps one could not be as positive about prestressed bars or wires placed inside a box section, as in the Lauffen-am-Neckar bridge,[34] or inside a tube. As far as the Author is aware, no example exists where the prestressing element has been grouted in. It would be reasonable to assume that the dangers of loss of prestress or failure due to shock loading are no greater for prestressed steelwork than for prestressed concrete and, in making this assertion, the Author concludes that the failure cited by Mr. Young was, in fact, in the latter material.

The questions on high-tensile bolts reflect the widespread interest which exists in this method of connection at the present time. Although experiments on such bolts were carried out in Great Britain a quarter of a century ago it was not until about eight years ago that American engineers began to use them, first in bridges and then in structures.

The current American specification is now fairly well-known here, some details having been published in *The Structural Engineer*.[60] This specification includes a clause that connections shall be designed as in riveted construction, rivets being replaced by the same number of H.T. bolts of the same diameter—a proviso which

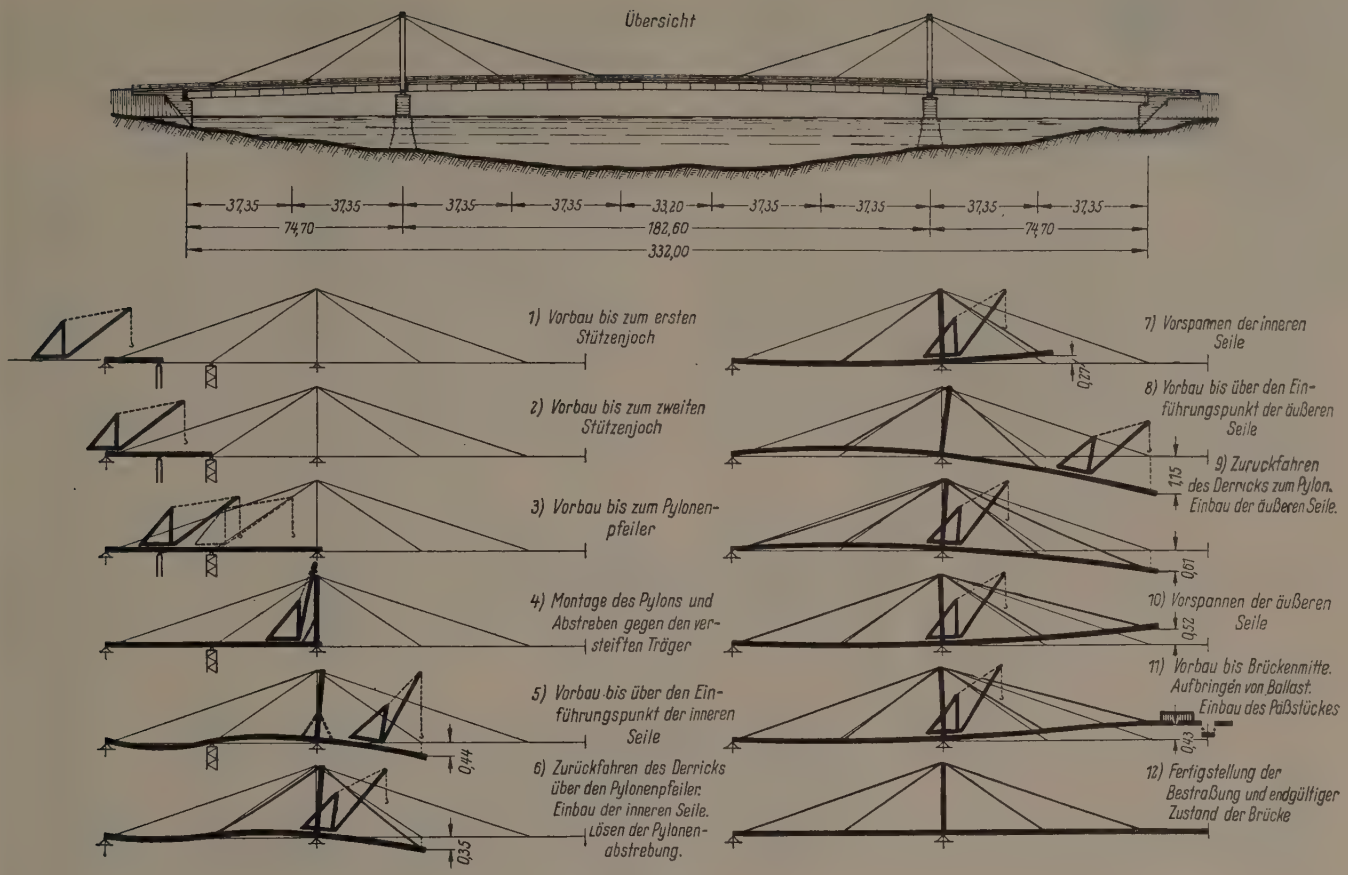


Fig. 9.

has been criticised here as being uneconomical and unscientific. Nevertheless, the Americans can justify this rule-of-thumb method of design. It has enabled them to get on with the job and it has saved them about 20 per cent in cost compared with riveting.

As Mr. Ranger and Mr. Kerensky emphasised, the initial German recommendations contained a similar clause. Indeed, the first German bridge employing H.T. bolts, a detail of which is shown in Fig. 16 of the paper, was designed in this manner.

Since the Author prepared his paper, however, the German Committee for Steel Structures has issued a most comprehensive " Preliminary Code of Practice " (which is, in effect, " out for comment ") thereby meeting the criticisms which have been levelled against the American document here.

Following further research work by Professor O. Steinhardt, 10 K bolts are specified (min. yield 57.2 tons/sq. in. ; min. U.T.S. 63.5 tons/sq. in.—*not* 58, as stated in the paper), with 6 S nuts (min. yield 30.5 tons/sq. in. ; min. U.T.S. 38.1 tons/sq. in.). Quite thin washers are now specified, but their hardness is the same as that of the bolts.

The contact surfaces must be flame-cleaned or sand-blasted within 5 hours of the connection being made, after which the coefficients of friction in mild or H.T. steel can be taken as 0.45 or 0.60 respectively.

In addition, separate factors of safety have been laid down for the design of structures and bridges or cranes, the factors being 1.25 and 1.60 respectively for normal loads (i.e. excluding wind) and 1.1 and 1.4 respectively when wind loads are included.

With these data, tables have been prepared for use in designing either structures or bridges and cranes in which the safe loads are given for various diameter bolts with single friction faces (analogous to single shear). A further table gives the assumed relationship between applied torque and induced tension.

By British standards, the German clearance of 1 mm. in the bolt-hole is small. Nevertheless, the Germans have assumed that the connections will act almost entirely by friction. Otherwise, they would not have included permissible stresses for bearing, which when calculated in the usual manner, are two-and-a-half times those normally used.

Returning now to Great Britain, both Mr. Young and Mr. Ranger had asked for comments on the current practice here. It is a fact that some failures have occurred and that suspicions have been aroused by H.T. bolts. The Author believes that most of the difficulties which have been experienced have been occasioned by the use of bolts and nuts complying with B.S. 1083, a Standard which was never intended to embrace H.T. bolts, but which has been used *faute de mieux*. It is quite common, for example, to specify

R bolts, as laid down in B.S. 1083, for which the steel has a U.T.S. of 45-55 tons/sq. in. and for which the engineer can demand a guaranteed yield of 34 tons/sq. in. For the corresponding nuts, however, B.S. 15 steel is used, with a guaranteed yield of 15.25 tons/sq. in. Consequently, it is not surprising that occasionally the threads of nuts have stripped.

At the present time, the whole problem is being considered by the B.S.I. In the meantime, the Author feels that steels of greater strength should be used here. For structures, the American type bolt, which is readily available, could be used. For bridges, the requirements vary, but in certain cases steels with a U.T.S. of 65-75 tons/sq. in. would be quite appropriate.

Additional Bibliography

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[57] Schäfer, G., "The New Highway Bridge over the Sava between Belgrade and Zemun (Yugoslavia)," *Acier-Stahl-Steel* (English Edition), May 1955.

[58] Beyer, E., and Ernst, H. J., "Erfahrungen und Seilversuche an einer seilunterspannten Verbundkonstruktion" ("Experiences and Cable Tests on a Cable-stressed Composite Structure"), *Der Stahlbau*, July 1957.

[59] Plum, A., "The New Motorway Bridge over the Rhine at Rodenkirchen (Germany)," *Acier-Stahl-Steel* (English Edition), June 1955.

[60] Easton, F. M., Lewis, E. M., and Wright, Dr. D. T., "Some Notes on the Use of High Preload Bolts in the United Kingdom," *The Structural Engineer*, May 1957.

Book Reviews

Stollen und Tunnelbau (Heading and Tunnelling), by W. Zanoskar. (Vienna, 1950 : Springer-Verlag.) 9 in. x 6 in. 231 pp. 74 fig. DM24.

More books appeared lately on the theory of tunnelling than on its practice, and many text-books are dated. The more welcome is the Author's record of much experience, partly collected under wartime conditions and correlated by a systematic mind. Dr. Zanoskar's purpose is eminently practical—to initiate the young graduate, the foreman and even the ganger into the intricacies of the job, to which they are often transferred from some different works. But expert engineers will be also interested in his notions on organization, on training of personnel, economy of works and cost accounting.

While the description of earlier methods is left to the historian ("a technical monument like the Mt. Cenis tunnel, blasted with gunpowder, cannot guide us any more"), the Author intentionally stresses some minor points, which look trivial but, if neglected, may jeopardize the efficiency of works. Though in charge of the controlling authority, he often acknowledges the ideas of contractors and of their staff, sometime he had to oppose directions given by the military. The greater part of the book refers to works in solid rocks, encountered in Austria and Norway. The model of a seven page progress record and a bibliography of German publications and very clean line drawings, instead of photographs, complete the book, which might be a help for learners and stimulating for practitioners.

A translation of a detailed table of contents, prepared by the reviewer, is in the Institution's Library, and will assist members with little knowledge of the German language to peruse numerical tables and especially nomograms.

P. M. T.

Modern Architecture in Brazil, by Henrique E. Mindlin. (London : Architectural Press, 1956.) 12 in. x 8½in. 256 plus xiii pp. 84s.

Building in this country, owing to financial stringencies following two world wars, is still basically utilitarian, and we have looked with envy to America and Scandinavia for examples of modern progressive design ; but these countries are being rapidly overhauled in adventurous and virile design as is well shown in this book, "Modern Architecture in Brazil," which is the first full scale attempt to show modern architecture in all its aspects in that country.

It is an inspiring book superbly illustrated with 450 beautiful photographs and 300 drawings, the survey comprising well over 100 examples drawn from all types of building including houses, blocks of flats, hotels, schools, churches, hospitals, public buildings, recreation centres and examples of landscape architecture.

In the first instance the well known work, "Brazil Builds," published by the Museum of Modern Arts in New York and now out of print, gained for Brazil's young architects immediate recognition in other countries. These brilliant designers have further developed their reputations for breathtaking brave new shapes in their projects. They have no need to give consideration in their designs to such elements as fog, rain and snow ; on the other hand the sun plays a striking part in the modelling and relief of their elevations. They have studied the phenomenon of sunlight and have created projective devices in the form of brise-soleil (permanent sun blinds), and have turned their attention to the plastic possibilities of reinforced concrete construction exemplified by large buildings supported on V columns.

This book is recommended to architects, structural engineers and students as an exemplar of progressive and virile design in the New World.

N. K.

Institution Notices and Proceedings

PRESIDENTIAL ADDRESS

Session 1957-58

A General Meeting of the Institution of Structural Engineers will be held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 3rd October, 1957, at 6 p.m., when Professor Sir Alfred Pugsley, O.B.E., D.Sc.(Eng.), F.R.S., M.I.Struct.E., M.I.C.E., F.R.Ae.S., will be installed as President for the Session 1957-58 and will give the Presidential Address.

FORTHCOMING MEETINGS

The following meetings have been arranged :

Thursday, October 24th, 1957

Ordinary General Meeting for the election of members 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when the following paper will be given : " Notes on the Use of High Preload Bolts in the United Kingdom " by Mr. F. M. Easton, A.M.I.Struct.E., A.M.I.C.E., Mr. E. M. Lewis, A.C.G.I., A.M.I.W., and Mr. D. T. Wright, B.A.Sc., M.S., Ph.D. (The paper was published in the May issue of the Journal.)

Thursday, October 31st, 1957

An Ordinary Meeting will be held at Church House, Westminster, S.W.1, at 6 p.m., when Mr. S. Barlow, M.I.Struct.E., and Mr. G. Foster, A.M.I.Mech.E., will give a paper on " Universal Beams and their Application." Afternoon tea will be served from 5.15 p.m. A very large attendance is expected at this meeting, including a number of members' guests, and in order to facilitate the arrangements for seating accommodation and for refreshments, members are requested to be good enough to apply to the Secretary for tickets by the 17th October.

Thursday, November 14th, 1957

Ordinary Meeting at 6 p.m., when Mr. F. R. Bullen, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E. (Honorary-Treasurer) will give a paper on " Structural Features of the new Binns Store in Middlesbrough."

Thursday, November 28th, 1957

Ordinary General Meeting for the election of members 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. L. A. Chackett, F.R.I.B.A., F.R.I.C.S., will give a paper on " The Relationship of the Architect and the Structural Engineer."

Thursday, December 12th, 1957

An Ordinary Meeting will be held at 6 p.m. for the presentation of the Maclachlan Lecture. The lecture, " Tandem Ferry Aprons for the Canadian Pacific Railway at Vancouver and Nanaimo, B.C., Canada," will be given by Mr. P. L. Easterbrook, B.Sc. (Graduate).

Thursday, December 19th, 1957

Ordinary General Meeting 5 p.m. This meeting is for the election of members and is open only to corporate members of the Institution.

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

With the exception of the meeting to be held at Church House, Westminster, on 31st October, all the above meetings will be held at the Institution's headquarters, 11, Upper Belgrave Street, London, S.W.1.

EXAMINATIONS, JANUARY, 1958

The Examinations of the Institution will next be held in the United Kingdom and Overseas on Tuesday and Wednesday, January 7th and 8th, 1958 (Graduateship) and January 9th and 10th (Associate-Membership).

MACLACHLAN LECTURE COMPETITION, 1958

The closing date for the receipt of entries for the next Maclachlan Lecture Competition is Monday, March 31st, 1958. The general conditions of the competition are as follows :

1. The Institution of Structural Engineers shall institute a written lecture to be known as the Maclachlan Lecture and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering as long as in every second year the subject shall be confined to steel structures.

3. Entrance into the competition for the Lecture shall be confined to Graduates and Associate-Members of the Institution who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award or for any other reason, the Institution shall transfer the above sums to the Research Fund of the Institution.

Particulars of the Competition for 1958

1. The Maclachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1958.

2. The subject of the Lecture may be on any aspect of Structural Engineering.

3. The work should be submitted as the script of a lecture which the Author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulae and detailed calculations should be avoided as far as possible in the text ; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Monday, March 31st, 1958.

DRURY MEDAL AWARD

The Sixth Competition for the Drury Medal Award will take place this year. The alternative subjects are as follows :

(*a*) The design of a bunker installation in a gravel pit.

(*b*) The design of a diving platform suitable for use in Olympic and National competitions.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary, envelope to be marked in the top left-hand corner " Drury Medal Award."

The closing date for the competition is October 1st, 1957.

The general conditions of the competition are as follows :

(1) The competition shall be for Graduates and Students of not more than 27 years of age.

(2) The subjects of the competition will be designs of a structural character, that is to say, involving structural design rather than planning.

(3) The subjects of design and the conditions shall be prepared and issued biennially.

(4) A Jury shall be appointed to examine the work submitted and to interview candidates if found necessary.

(5) In order to ensure that the design submitted is the unaided work of the competitor, the drawings, calculations etc., submitted shall be endorsed by the candidate : " I declare that the work I hereby submit is my own unaided work." The declaration shall be signed by the competitor, and be either countersigned by a corporate member, or be certified as made before a Justice of the Peace, or a Commissioner for Oaths.

KING GEORGE VI MEMORIAL FELLOWSHIPS

It is announced that King George VI Memoria Fellowships, sponsored by the English-Speaking Union of the United States, are once again being offered to British students to enable them to study in American universities. The Fellowships are for students in Technology and the Applied Sciences and are available not only for University graduates but also for holders of the Higher National Certificate. Up to 30 awards are given each year and are divided between the two categories.

Candidates for Fellowships for the academic year 1958-59 must be :

(a) Citizens of Great Britain or Northern Ireland ;

(b) Holders of a British University degree or of a Higher National Certificate, or candidates for such a degree or Certificate in the summer of 1958 ;

(c) Not less than 18 nor more than 30 on August 31st, 1958 ;

(d) Unmarried.

Full particulars and application forms may be obtained from Mrs. D. R. Dalton, B.A., King George VI Memorial Fellowship Committee, 37, Charles Street, Berkeley Square, London, W.1. Completed forms must be submitted not later than November 16th, 1957.

When enquiring about the awards, students are asked to state the qualifications they have or expect to take ; the British University or College at which they have studied ; their proposed field of study in the United States ; their date of birth and nationality and marital status.

LONDON GRADUATES' AND STUDENTS' SECTION

A meeting was held in conjunction with the Architectural Association at Bedford Square, W.C.1, on the 16th May. Mr. Wolstenholme gave a short introductory description of a cantilever roof to a filling station on the sea front at Seabrooke, and this was followed by a general discussion.

On the 22nd June, a party of twenty-nine members and friends visited the Harwell Atomic Energy Research Establishment. A short introductory talk and film were given in the Cockcroft Hall, and the party was then taken to see the laboratories and the atomic pile.

The opening meeting of the Session will be held at 11, Upper Belgrave Street, London, S.W.1, on Wednesday, 23rd October, at 6.30 p.m. This will be a discussion meeting on the subject " That the Architect should be replaced by the Engineer in regard to industrial structures." The speakers will be Mr. F. J. Samuely, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., and an Architect.

A meeting will be held on Monday, 18th November, at 6.30 p.m., when Mr. J. K. Anderson, M.A., M.I.C.E., will give a talk on " the Forth Bridge."

On Tuesday, 10th December, a meeting will be held at which papers by members of the Section will be presented.

Hon. Secretary : A. S. Beeson, 28, Arnold Crescent, Whitton Dene, Isleworth, Middlesex.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :

Thursday, October 17th, 1957

The inaugural meeting of the Session will be held in Manchester, commencing at 7.15 p.m., at which the Chairman's Address will be given by Mr. W. Fitton, B.Sc.(Eng.), M.I.Struct.E., A.M.I.Mech.E. A film will be shown on " The Dounreay Sphere." (A coach will run from Liverpool.)

Monday, November 11th, 1957

Mr. G. B. Godfrey, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E. (Associate-Member of Council) on " Steel Tubes and Hollow Sections."

Monday, December 16th, 1957

Mr. S. Richards, A.M.I.Struct.E., and Mr. N. Lancaster : " A Review of the Structural Steelwork in the ' Seraphim ' Extensions at Appleby-Frodingham."

Unless otherwise stated all lectures will commence at 6.30 p.m. and will be preceded by light refreshments. All the above meetings will take place at the College of Science and Technology, Manchester.

MERSEYSIDE PANEL

The following meeting has been arranged :

Tuesday, November 12th, 1957

Mr. G. B. Godfrey, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E. (Associate-Member of Council) on " Steel Tubes and Hollow Sections." At the Liverpool Engineering Society, 9 The Temple, 24 Dale Street, Liverpool, 2. Light refreshments will be available from 6 p.m., and the meeting will commence at 6.45 p.m.

Joint Hon. Secretaries : J. L. Robinson, A.M.I.Struct.E., 314, Northenden Road, Sale, Manchester. M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford, 6.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :

Saturday, October 12th, 1957

The Annual Dinner and Ladies' Evening will be held in the Botanical Gardens, Birmingham.

Friday, October 25th, 1957

The Chairman's Address will be presented by Mr. L. J. Griffiths, M.I.Struct.E., at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m.

Wednesday, November 13th, 1957

Mr. F. I. Lees, B.Sc.(Eng.) on " The Fabrication and Erection of Pelham Bridge, Lincoln " at the Electricity Demonstration Hall, Irongate, Derby, at 6.15 p.m.

Friday, November 22nd, 1957

Mr. H. C. Husband, B.Eng., M.I.Struct.E., M.I.C.E., M.I.Mech.E., Honorary Curator and Mr. K. H. Best, B.Eng., M.I.Struct.E., A.M.I.C.E. on " Some Recent Highway Bridges in Ceylon." At the James Watt Memorial Institute, Great Charles Street, Birmingham, 6 p.m.

Hon. Secretary : J. R. Chaffer, M.I.Struct.E., 107, Jockey Road, Sutton Coldfield, Warwickshire.

GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :

Friday, November 1st, 1957

Mr. L. G. Booth, Ph.D., M.A., on " Structural Use of Timber."

Friday, December 6th, 1957

Joint meeting with the Junior Section of the Institution of Municipal Engineers. Mr. W. J. Young, B.A.I., on " The Cement Gun and its Use in the General Field of Structural Engineering."

Both meetings will be held at the Birmingham Exchange and Engineering Centre, Stephenson Place, Birmingham, at 6.30 p.m.

Hon. Secretary : F. A. Butterworth, " Roscrea," Tansey Green, Pensnett, Brierley Hill, Staffs.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :

Monday, October 21st, 1957

At Newcastle. Installation of Chairman. Mr. D. M. O'Herlihy, O.B.E., B.Sc., M.I.Struct.E., M.I.C.E., will give the Chairman's Address on " Municipal Works at Tynemouth."

Tuesday, October 22nd, 1957

At Middlesbrough. The Chairman, Mr. D. M. O'Herlihy, O.B.E., B.Sc., M.I.Struct.E., M.I.C.E., will give a paper entitled " Municipal Works at Tynemouth."

Thursday, November 7th, 1957

At Middlesbrough. A joint meeting with the Northern Architectural Association will be held at which Mr. E. Shepley, B.Sc., M.I.C.E., will lecture on " Choice of Structure."

Wednesday, November 20th, 1957

At Middlesbrough. Joint Management Meeting at which a lecture will be given by Mr. Peter G. Masefield.

Thursday, November 21st, 1957

At Newcastle. A lecture will be given by Sir F. C. Hooper at a Joint Management Meeting to be held in the Connaught Hall, Blackett Street, Newcastle.

Tuesday, December 3rd, 1957

At Middlesbrough.

Wednesday, December 4th, 1957

At Newcastle.

Mr. F. M. Bowen, M.I.Struct.E., M.I.C.E., Assoc.I.Mech.E. (Member of the Council) and Mr. B. E. S. Ranger, A.M.I.Struct.E., A.M.I.C.E. (Associate-Member of Council) on " Steelwork in Transit Shed 102, Southampton Docks."

Meetings in Middlesbrough will be held in the Cleveland Scientific and Technical Institution and meetings in Newcastle (except where noted) in the Neville Hall.

All meetings commence at 6.30 p.m., preceded by buffet tea at 6 p.m.

Hon. Secretary : H. W. Dowe, A.M.I.Struct.E., 143, Laburnum Road, Redcar, Yorks.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :

Tuesday, October 1st, 1957

Mr. W. H. Chilton, A.M.I.Struct.E., on " Some Structural Aspects of Nuclear Reactors " at the College of Technology,,Belfast, 6.45 p.m.

Tuesday, November 5th, 1957

Details to be arranged.

Monday, December 9th, 1957

Meeting to be held with the Northern Ireland Association of the Institution of Civil Engineers at Queen's University, Belfast. Professor J. L. Montrose on " The Interpretation of Contracts and Arbitration."

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., " Barbizon," 26 Dunlambert Park, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :

Monday, October 7th, 1957

At the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, at 7 p.m. Chairman's Address by Mr. G. M. Dingwall, M.I.Struct.E., M.I.C.E.

Tuesday, October 8th, 1957

Annual Dinner and Dance at the Grosvenor Restaurant, Gordon Street, Glasgow, 7 p.m.

Tuesday, October 29th, 1957

At the Institution of Engineers and Shipbuilders, Glasgow, 7 p.m. Mr. T. Whitley Moran, B.A., B.A.I., M.I.Struct.E., M.I.C.E., on " The Use of Gunite as a Structural Material."

Tuesday, November 5th, 1957

At the Adelphi Hotel, Coburn Street, Edinburgh, 6.15 p.m. Mr. F. M. Bowen, M.I.Struct.E., M.I.C.E., Assoc.I.Mech.E. (Member of Council) on " Steelwork in Transit Shed 102, Southampton Docks."

Wednesday, November 20th, 1957

At the Institution of Engineers and Shipbuilders, Glasgow, 7 p.m. Joint Meeting with the West of Scotland Branch of the Institute of Welding. Mr. J. A. Forrest, A.M.I.Mech.E., and Mr. James McLean, B.Sc., on ".Fabrication, Erection and Welding Dounreay Sphere."

Tuesday, December 10th, 1957

At the Institution of Engineers and Shipbuilders, Glasgow, 7 p.m. Mr. O. A. Kerensky, B.Sc., M.I.Struct.E., M.I.C.E., on "Design of Plate Girders."

Hon. Secretary: W. G. Cantlay, B.Sc.(Eng.), A.M.I.Struct.E., A.M.I.C.E., 3, Blairbeth Terrace, Burnside, Glasgow.

SOUTH WESTERN COUNTIES BRANCH

The following meetings have been arranged:

Friday, November 1st, 1957

At the Duke of Cornwall Hotel, Plymouth, 7 p.m. Chairman's Address by Mr. E. W. Howells, M.I.Struct.E. The President and Secretary of the Institution will attend the meeting which will be followed by an informal Dinner.

Friday, December 6th, 1957

At the Duke of Cornwall Hotel, Plymouth, 6 p.m. (Tea will be served at 5.30 p.m.) Mr. Clifford E. Saunders, M.I.Struct.E., A.M.Inst.W., on "Some Unusual Structural Engineering Problems."

Hon. Secretary: C. J. Woodrow, J.P., "Elstow," Hartley Park Villas, Mannamead, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged:

Wednesday, October 30th, 1957

At the Mackworth Hotel, Swansea, 6.30 p.m. Chairman's Address, "Safety in Construction," by Mr. G. R. Brueton, M.I.Struct.E., A.M.I.C.E.

Tuesday, November 5th, 1957

The Chairman's Address will be repeated and will be followed by a discussion.

At the South Wales Institute of Engineers, Park Place, Cardiff, 6.30 p.m.

Saturday, November 9th, 1957

At the County Buildings, Colwyn Bay, 6 p.m. The Chairman's Address will be repeated and will be followed by a discussion.

Tuesday, December 3rd, 1957

At the South Wales Institute of Engineers, Cardiff, 6.30 p.m. Joint Meeting with the Institute of Welding. Mr. A. V. Hooker, M.I.Struct.E., M.I.C.E. (Past Chairman), "Recent Thoughts on Structural Welding."

Wednesday, December 11th, 1957

At the Mackworth Hotel, Swansea. Joint Meeting with the Institution of Civil Engineers. Details to be announced.

Hon. Secretary: K. J. Stewart, A.M.I.Struct.E., A.M.I.C.E., 15, Glanmor Road, Swansea.

WESTERN COUNTIES BRANCH

The following meetings have been arranged:

Friday, October 4th, 1957

Chairman's Address by Mr. Clifford E. Saunders, M.I.Struct.E.

Friday, November 1st, 1957

Combined Meeting with the Institution of Civil Engineers. Mr. J. H. G. King, A.M.I.Struct.E., A.M.I.C.E., on "A Guide to Foundation Problems through Geology."

Friday, November 15th, 1957

Three papers to be given for the special interest of the junior members:

Mr. H. G. Gooding, A.R.I.C.S., on "Final Accounts."

Miss M. Woodmancy (Student) on "Trends in Modern Architecture."

Mr. R. Purnell (Graduate) on "Precast Concrete Construction."

Friday, December 6th, 1957

Mr. R. G. Taylor, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., on "Tubular Structures."

The above meetings will be held in the Small Lecture Theatre, University Engineering Laboratories, University of Bristol, at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary: E. Hughes, M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The following meetings have been arranged:

Wednesday, October 16th, 1957

At Leeds. Chairman's Address by Mr. T. F. Cliffe, M.I.Struct.E., A.R.I.C.S.

Tuesday, October 17th, 1957

The Chairman's Address will be repeated at Sheffield.

Wednesday, November 20th, 1957

At Leeds. Joint Meeting with the Yorkshire Association of the Institution of Civil Engineers. Mr. H. C. Husband, B.Eng., M.I.Struct.E., M.I.C.E., M.I.Mech.E. (Hon. Curator) and Mr. K. H. Best, B.Eng., M.I.Struct.E., A.M.I.C.E. on "Recent Highway Bridge Construction in Ceylon."

Wednesday, December 11th, 1957

At Leeds. Mr. G. B. Godfrey, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E. (Associate-Member of Council) on "Post-War Developments in German Steel Bridges and Structures."

Thursday, December 12th, 1957

The above meeting will be repeated at Sheffield.

Meetings at Leeds will be held at the Central Electricity Authority Offices, Whitehall Road, at 6.30 p.m.

Meetings at Sheffield will be held at the Royal Victoria Hotel, at 6.30 p.m.

Hon. Secretary: W. B. Stock, A.M.I.Struct.E., 34, Hobart Road, Dewsbury, Yorks.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary: A. E. Tait, B.Sc., A.M.I.Struct.E., A.M.I.C.E., P.O. Box 3306, Johannesburg, South Africa.

During weekdays Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. Phone 34-1111, Ext. 257.

Natal Section Hon. Secretary: J. C. Panton, A.M.I.Struct.E., A.M.I.C.E., c/o Dorman Long (Africa) Ltd., P.O. Box 932, Durban.

Cape Section Hon. Secretary: R. F. Norris, A.M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

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Hon. Secretary: K. C. Davey, A.M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

SINGAPORE AND FEDERATION OF MALAYA SECTION

Chairman: T. Karmakar, M.I.Struct.E.

Hon. Secretary: W. N. Cursiter, B.Sc., A.M.I.Struct.E., c/o Redpath Brown and Co. Ltd., P.O. Box 648, Singapore.

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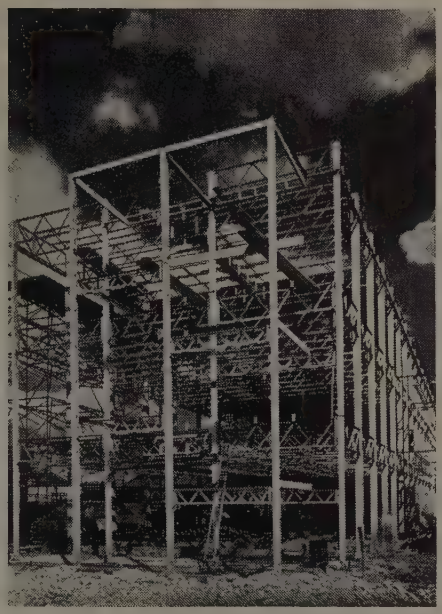

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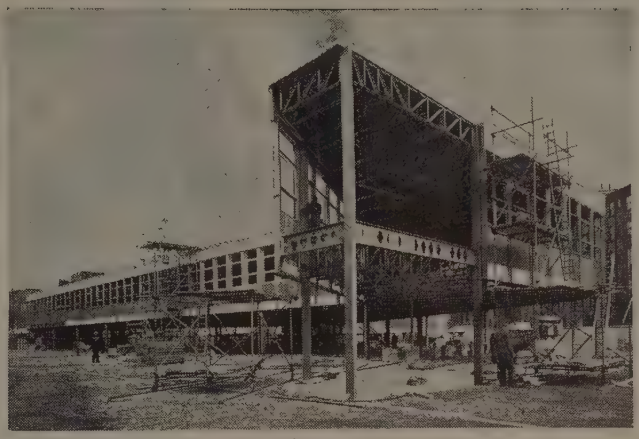
Architect : E. Firth, F.R.I.B.A., A.M.T.P.I., City Architect.



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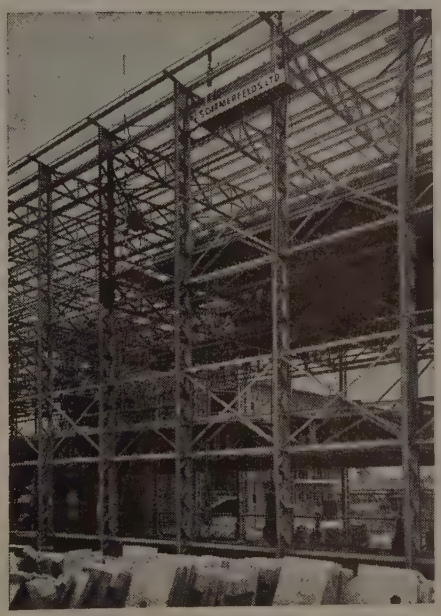

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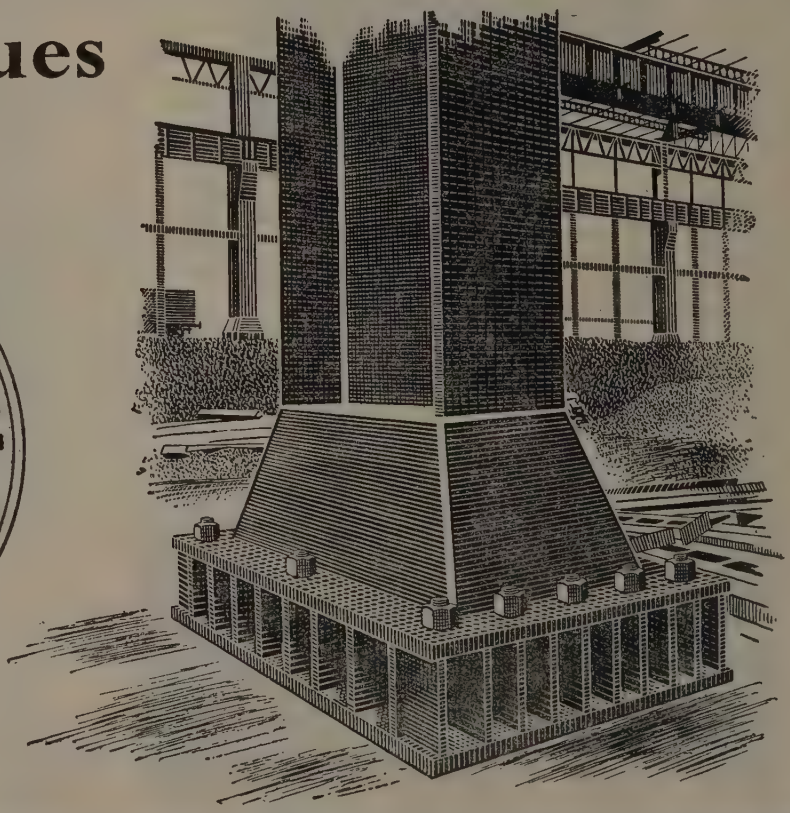

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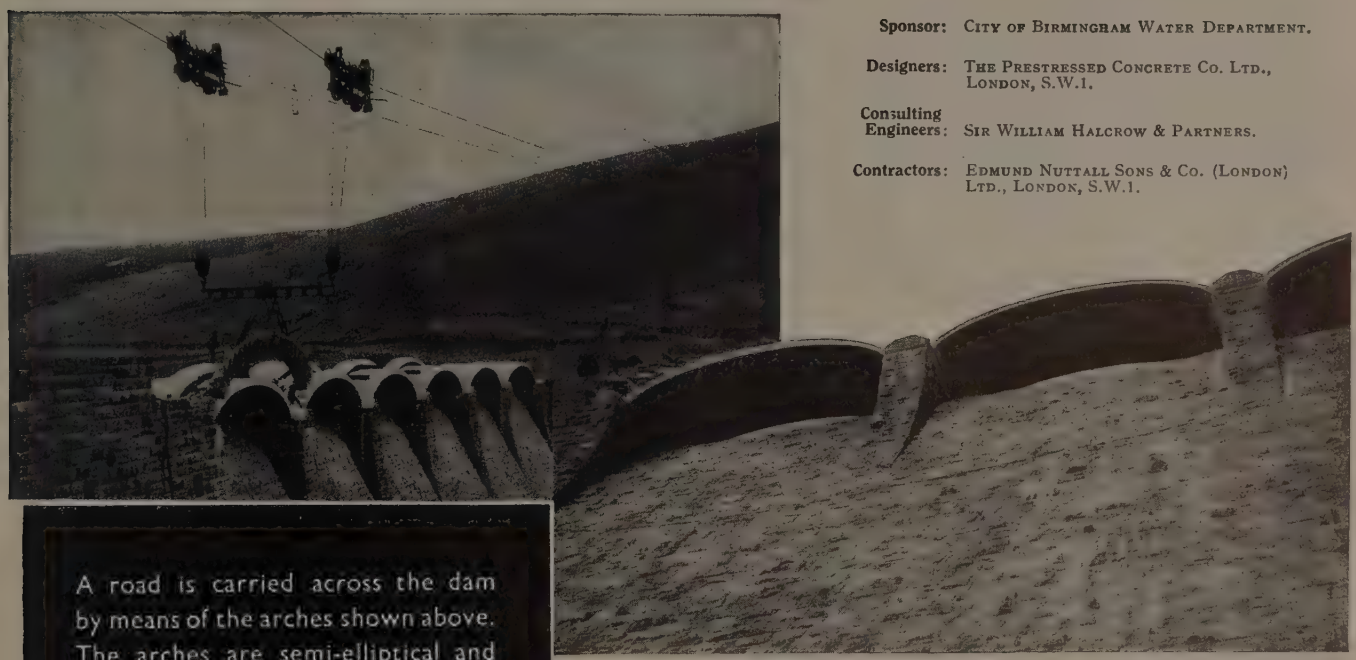

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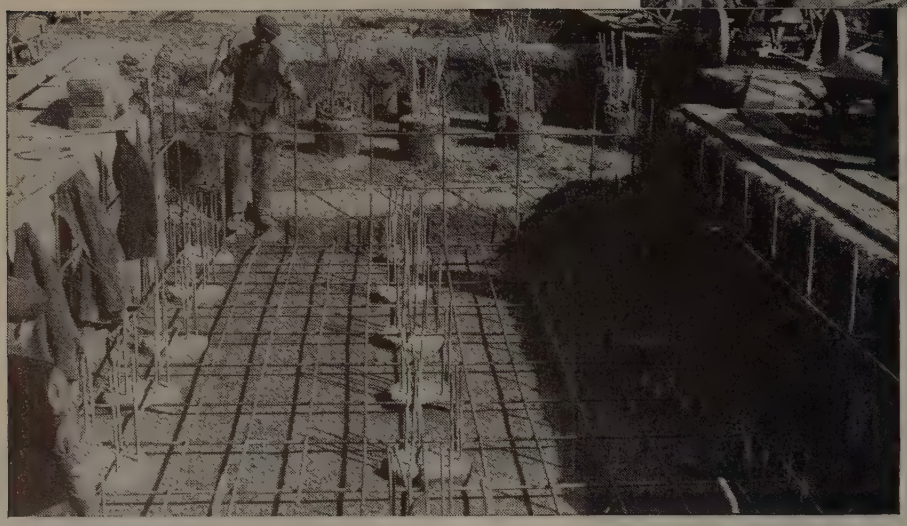

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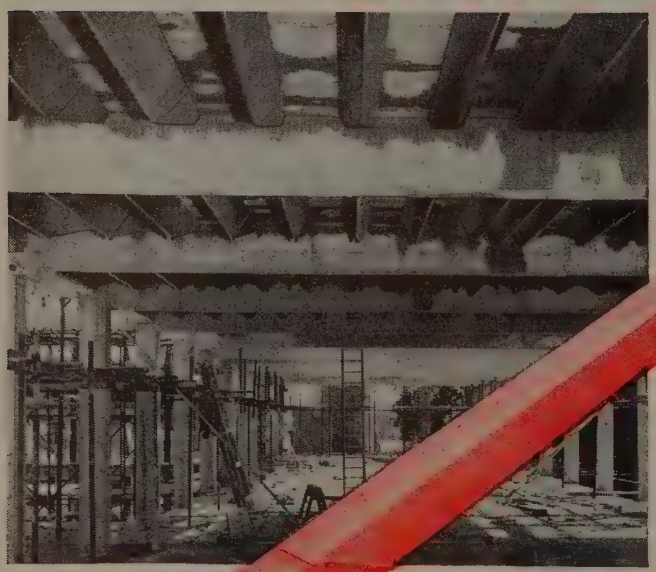

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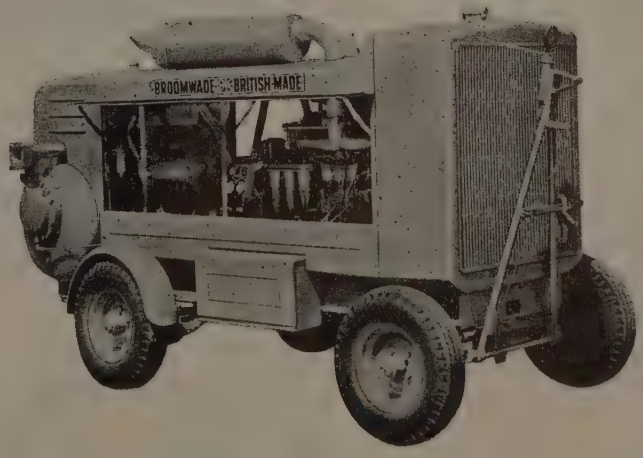

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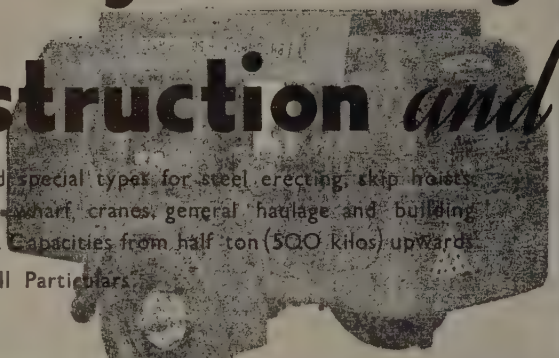

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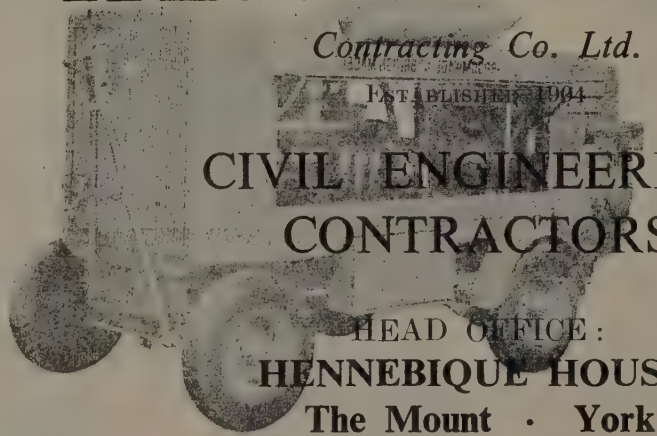

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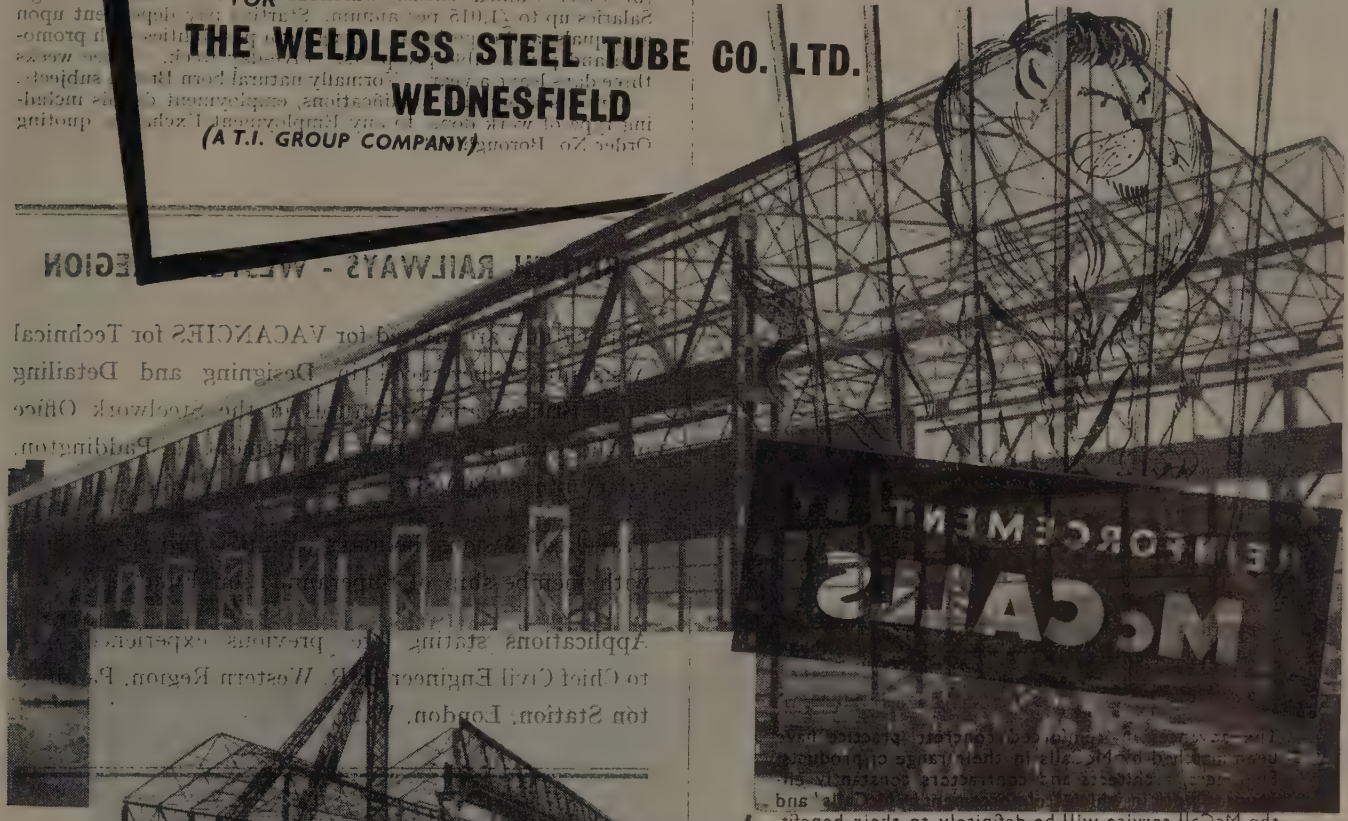

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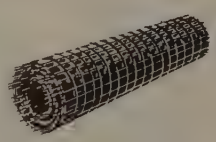

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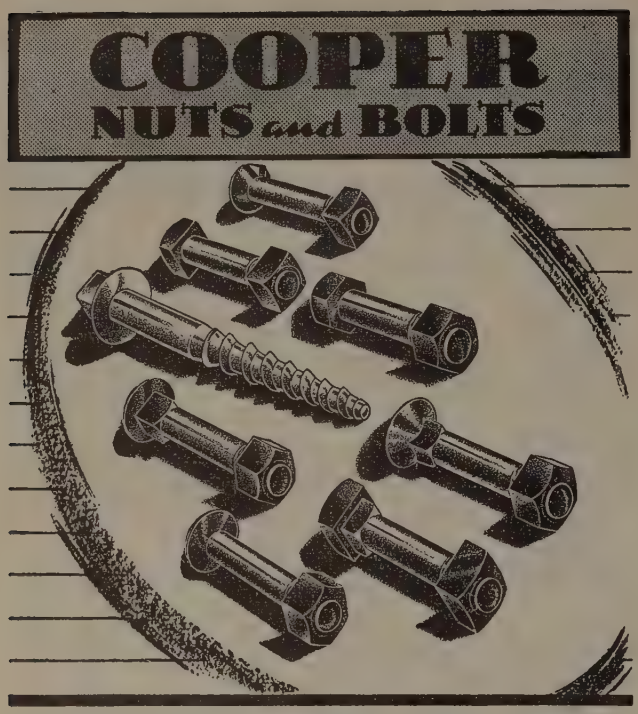

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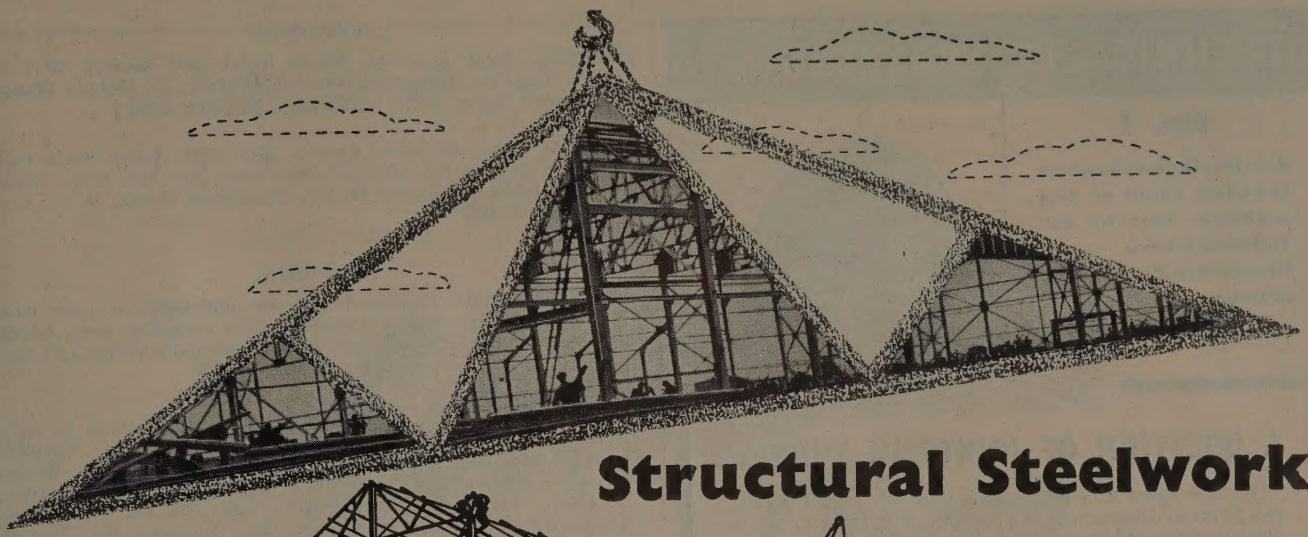

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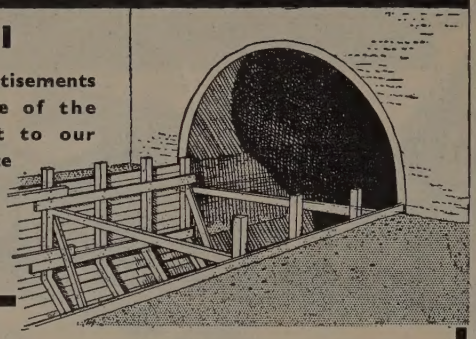

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